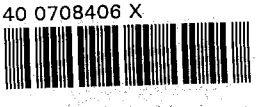
FOR REFERENCE ONLY

21 JUN 2000

40 0708406 X



FLOW ROUTING IN THE PARALLEL PIPES OF A COMBINED SEWER SYSTEM

by

Wai Keung AU YEUNG

Thesis submitted to the Council for National Academic Awards in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

Department of Civil Engineering,
Surveying and Building,
Dundee Institute of Technology, Dundee
in collaboration with
Trent Polytechnic, Nottingham, and
Fife Regional Council.

February 1990

ProQuest Number: 10183063

All rights reserved

INFORMATION TO ALL USERS
The quality of this reproduction is dependent upon the quality of the copy submitted.

In the unlikely event that the author did not send a complete manuscript
and there are missing pages, these will be noted. Also, if material had to be removed,
a note will indicate the deletion.



ProQuest 10183063

Published by ProQuest LLC(2016). Copyright of the Dissertation is held by the Author.

All rights reserved.
This work is protected against unauthorized copying under Title 17, United States Code.
Microform Edition © ProQuest LLC.

ProQuest LLC
789 East Eisenhower Parkway
P.O. Box 1346
Ann Arbor, MI 48106-1346

ABSTRACT

FLOW ROUTING IN THE PARALLEL PIPES OF A COMBINED SEWER SYSTEM

by W.K. AU-YEUNG

A parallel pipe model has been constructed and successfully applied to a twin pipe sewer system. The model predicts flow and level for each of the parallel pipes and deals with the complex behaviour which occurs at cross-connections.

The model DUPPERS is an enhancement of the single pipe simulation model DUCTS developed at Dundee Institute of Technology. It has been rigorously tested using a variety of artifical parallel pipe system configurations and input discharges. Obtained results showed that the model was operating satisfactorily. The model was applied to the Lyneburn system in Dunfermline, Scotland, with three models being constructed : two sub-models and a full system model. Comparison between observed and predicted hydrographs for both flow and level were in close agreement. Percentage differences for peak flowrates, runoff volumes and levels were all within ±20% of observed and simulated values.

A review of flow routing procedures, above-ground hydrological models and the major commercial sewer simulation packages is also included.

In addition to the tailored parallel pipe model, single combined pipe models for the system based upon the commercial package WASSP and the in-house model DUCTS were also constructed. The simulation outputs from these two models were found to be close to the observed flow data. However, the lumped pipe models only predicted the combined flow for the twin pipe outfalls. The construction of these models verified that the Sewered Sub-Area model could be applied to the study catchment. Furthermore, the model DUCTS was shown to perform satisfactorily. Catchment data such as contributing areas, sub-catchment type and paved areas used in the lumped pipe models were subsequently used in the parallel pipe model.

The data collection techniques are described in detail. A sequential flow logging procedure was used and found to be an effective and economic method of data collection, especially where a limited number of loggers was available.

The model was successfully used to examine the storm runoff for the Lyneburn parallel pipe system and also the complicated hydraulic behaviour in cross-connections. The numbering system for the parallel pipe networks, the chosen equations to represent the different flow patterns for cross-connections, together with the level computation procedure form the major enhancements incorporated into DUPPERS.

CONTENTS

		Page
ACKNOWLEDGEMENTS		ix
LIST OF TABLES		xi
LIST OF FIGURES		xv
LIST OF PLATES		xxiv
NOTATION		xxvi
GLOSSARY OF TERMS		xxvii
1.	INTRODUCTION	1
	1.1 General Introduction	1
	1.2 Research Aims and Objectives	4
	1.3 Research Presentation	6
2.	DEVELOPMENT OF URBAN DRAINAGE RUNOFF MODELLING IN THE UK	8
	2.1 Introduction	8
	2.2 Precipitation	9
	2.2.1 Rainfall Depth-Duration-Frequency Relationships	10
	2.2.2 Storm Profiles	11
	2.2.3 Storm Movement	12
	2.3 Overland Surface Flow	13
	2.3.1 Depression Storage	14
	2.3.2 Runoff Volume	15
	2.3.3 Surface Routing	16
	2.4 Sewer Flow Routing	17
	2.4.1 Free-Surface Flow Routing	18
	2.4.2 Pressurised Pipe Routing	19
	2.5 The Framework Of DUPPERS	20
3.	SEWER FLOW ROUTING METHODS	26
	3.1 Introduction	26
	3.2 The Unsteady Flow Equations	27
	3.2.1 The Saint-Venant Equations	28
	3.2.2 Simplified Methods Of Solution	29

3.3 Numerical Methods 32

3.3.1 Methods Of Characteristics 33
3.3.2 Explicit Methods 36
3.3.3 Implicit Methods 39
3.3.4 Comparison Of Numerical Methods 42

3.4 Muskingum-Cunge Method 43

3.4.1 Basic Muskingum Theory 44
3.4.2 Improvements By Cunge 45
3.4.3 Choice Of Muskingum-Cunge Parameters For Pipe Flow Routing 47

3.5 Review Of Flow Routing Methods 49

4. CONCEPTUAL SIMULATION MODELS OF URBAN CATCHMENTS 55

4.1 Introduction 55

4.2 Chicago Hydrograph Method 57

4.3 University Of Cincinnati Urban Runoff Model (UCURM) 58

4.4 Transport And Road Research Laboratory Hydrograph Method (TRRL) 59

4.5 Illinois Urban Drainage Area Simulator (ILLUDAS) 60

4.6 Storm Water Management Model (SWMM) 61

4.7 S - 11 - S 63

4.8 Wallingford Procedure Storm Simulation Package (WASSP) 64

4.9 Dundee College Of Technology Sewerage Model (DUCTS) 67

4.10 The Need For DUPPERS 69

5. STUDY CATCHMENT 74

5.1 Introduction 74

5.2 Lyneburn Sewerage System 75

5.3 Catchment Survey 76

5.3.1 Above Ground Survey 77
5.3.2 Below Ground Sewer Survey 78

5.4 Characteristics Of Lyneburn Drainage System 80

5.4.1 Contributing Areas 80
5.4.2 Parallel Pipes 81
5.4.3 Cross-Connection Overflows 82

6. INSTRUMENTATION AND DATA COLLECTION 99

6.1 Introduction 99

6.2 Measurement Of Rainfall 100

6.2.1 Raingauges And Data Loggers 102
6.2.2 Raingauge Calibration 103

6.3 Flow Monitoring Equipment 105

6.3.1 In-Sewer Flow Loggers 106
6.3.2 Flow Determination 107
6.3.3 Logger Calibration Under Free Surface Flow 108
6.3.4 Logger Calibration Under Surcharged FLow 110
6.3.5 General Accuracy Of Flow Loggers 110

6.4 Catchment Data Capture and Processing 111

6.4.1 Equipment Locations 112
6.4.2 Duration Of Installations 115
6.4.3 Data Management 116
6.4.4 Data Accuracy And Consistency Checks 118
6.4.5 Available Captured Events 119

6.5 Interpretation Of Rainfall & Runoff Data 120

6.5.1 Sewer Roughness Value 120
6.5.2 Dry Weather Flows 122
6.5.3 Depression Storage And Runoff Coefficient 123
6.5.4 Percentage Runoff 124
6.5.5 Flows At Cross-Connections 125

7. SINGLE PIPE MODEL AND SYSTEM SIMPLIFICATION 153

7.1 Introduction 153

7.2 Study Catchment Simplification 155

7.2.1 Sub-Catchments For Simplification 156
7.2.2 Comparison Of Simplified Models 156
7.2.3 Adoption Of Sewered Sub-Area Model 158

7.3 Equivalent Single Pipe Model 159

7.3.1 Adoption Of WASSP 160
7.3.2 Adoption Of DUPPERS 160

7.4 Comparison Of Results 161

8. A PARALLEL PIPE MODEL 182

8.1 Introduction 182

8.2 DUPPERS Parallel Pipe Model 183

8.3 Flow Continuity 184

8.3.1 Straight Through Flows 185
8.3.2 Overflow 186
8.3.3 Head Balance 186
8.3.4 Reverse Flow 187

8.4 Hydraulic Aspects Of Flows At Cross-Connections 188

8.4.1 Analysis Of Side Weir Discharge 190
8.4.2 Overflow Computation At Cross-Connections 192
8.4.3 Coefficient Of Discharge 194

8.5 Flow Depth Calculation 195

9. DEVELOPMENT OF THE DUPPERS SIMULATION MODEL 201

9.1 Introduction 201

9.2 Enhancement To Ducts 202

9.2.1 Computing Strategy 203
9.2.2 Parallel Pipes 205
9.2.3 Cross-Connection 206
9.2.4 System Data Input And Output 207
9.2.5 Parameter Storage 208

9.3 Computing Procedure For Flows At Cross-Connection 210

9.3.1 Free Surface Overflow 211
9.3.2 Pressurised Overflow 212
9.3.3 Flow Balance 213
9.3.4 Reverse Flow 214

9.4 Management Of DUPPERS Parallel Pipe Model 215

9.4.1 Required Input Data 215
9.4.2 Level Computation 216
9.4.3 Flow Routing In Parallel Pipes 218

9.5 Operation of DUPPERS 219

9.6 Limitations Of The Model 220

10. SIMULATION MODEL ACCURACY ASSESSMENT 235

10.1 Introduction 235

10.2 Input Data For Testing 236

10.2.1 Test Systems 236
10.2.2 Test Inputs 237

10.3 Testing Components Of The Model 238

10.3.1 Free Surface Flow 238
10.3.2 Pressurised Flow 241
10.3.3 Transitional Flow 243
10.3.4 Level Prediction 244
10.3.5 Head Above Weir 245
10.3.6 Overflow / Reverse Flow And Level Balance 246
10.3.7 Continuity Of Discharge 248
10.3.8 Varying Intensity Rainfalls 249
10.3.9 Conclusions From Model Components Testing 250

10.4 Performance Of Lyneburn Simulation Model 251

10.4.1 Performance Under Steady State Conditions 253
10.4.2 Model Calibration And Verification 255

10.5 Analysis Of Model Performance 258

10.5.1 Peak Discharge 259
10.5.2 Runoff Volume In Parallel Pipes 261
10.5.3 Runoff Volume Between Various Locations 262
10.5.4 Percentage Runoff 264
10.5.5 Level Of Reliability 265

11. OVERVIEW OF THE MODEL AND CONCLUSIONS 331

11.1 Introduction 331

11.2 Appraisal Of DUPPERS Model 331

11.3 General Conclusions 333

11.4 Conclusions On Simulation Results 335

11.5 Suggestions For Further Work 336

REFERENCES 338

APPENDICES

A. Presentation Of Computer Program DUPPERS 360

B. Computer Listing Of The Graphical Plotting Program GPLOT 416

C. Comparison Of Observed And Predicted Flow (By DUPPERS) For Parallel Pipes At Catchment Outfalls 424

D. Methodology Of Combining Parallel Pipes Into A Single Equivalent Pipe Diameter 445

E. Longitudinal Section Of Lyneburn Parallel Sewers With Geometric Characteristics 450

F. Standard Booking Forms For Sewer Survey And For The Equipment Records In The Lyneburn Drainage System Study 454

G. Presented Paper 460

(i) The Lyneburn Case Study 461
(ii) The Development of Flow Simulation Models for Urban Catchments in North East Scotland 464

ACKNOWLEDGEMENTS

The Research described in this thesis was carried out in the Civil Engineering Department of Dundee Institute of Technology between 1985 and 1989. Throughout the whole research period, advice and help were obtained from many sources and individuals and therefore it is not quite possible to name every one of them. However, the author wishes to express his gratitude to the following persons and organisations :

Dr. C. J. Pratt for acting as director of studies and being an external supervisor, never failed to keep up the momentum and guidance throughout the duration of the research project.

Mr. C. Jefferies for acting as direct supervisor for the work, and for the encouragement and guidance throughout the whole research duration. Thanks also given for reading and correcting the thesis in such a faultless way.

Mr. R. M. Ashley for acting as second supervisor, and for the valuable advice from time to time throughout the research duration.

Mr. G. S. W. Stevens, group leader of drainage department in Fife Regional Council, for his assistance in the sewer survey and advice on the complicated study sewerage system.

Fife Regional Council, for supplying the electrical and electronic equipment for capturing both rainfall and flow data; and for the permission to use their computer.

The sewer gang in Dunfermline district, for their support during underground sewer survey and acting as safety support.

Thanks have also given to the departmental technicians, specially Mr. J. Crawford, Mr. R. Peter and Mr. G. Gerrard for their assistance in sewer survey work and the maintenance of the data logging equipment.

Mr. S. Gardner, Computer Centre programmer, for his advice in solving the computational problems.

Mr. W. King and Mr. Robertson, local residents in Dunfermline, for their kind permission for installing raingauges in their gardens.

The author also wants to express his gratitude to Keith AU YEUNG, for the developement of the first version of the graphical plotting program GPLOT in DEC mainframe computer.

Finally to Yvonne and Louise, my wife and daughter, for the moral support throughout the entire research duration.

LIST OF TABLES

		Page
5.1	Summary of Sub-Catchment Type and Their Catchment Characteristics	84
5.2	Lengths and Size of Contributing Areas for the Study Catchment Main Sewers	84
5.3	Hydraulic Characteristics and Weir Dimensions for the Five Cross-Connections and Overflows Shown in Figure 5.9	85
6.1	Locations of Raingauge and Logger and Their Descriptions	127
6.2	Logger Locations and Duration of Monitoring	128
6.3	Comparison of Total Rainfall Captured at the Two Raingauge Sites	129
6.4	Catchments with the Available Events	129
6.5	Dry Weather Flows at Flow Monitoring Locations	130
6.6	Values of DEPSTOG and RC for Flow Monitoring Locations	130
7.1	Sub-catchment Details for Scotland Drive and Garvock Bank	163
7.2	Observed Events and Simulation Models Chosen for the Five Catchments	164
7.3	Peak Discharges and Runoff Volumes of WASSP and Observed Values	164
7.4	Peak Flows and Runoff Volumes for All Models	165

9.1 Descriptions of the Major Subroutines in DUPPERS and the Details of Enhancements 223

9.2 Condition Statements for Free-Surface and Pressurised Overflows 224

9.3 Condition Statements for Head Balance and Reverse Flow Conditions 224

9.4 Branch Number and Ancillary Index Values 225

9.5 Details of Required Data for Parallel Pipe System 225

9.6 Input Descriptions and Format for Parallel Pipes, CCOs and Bridging Pipes 226

10.1 Details of the Two Testing Parallel Pipe Systems 268

10.2 Pipe Diameters and Corresponding Test Objections 269

10.3 Dimensions and Hydraulic Details of Parallel Pipes and Cross-Connections in Test Systems A and B 270

10.4 Uniform Rainfall Intensities and Corresponding Inflows to the Parallel Pipe Systems 271

10.5 Simulated Outputs for System A under Free-Surface Flow Conditions 271

10.6 Simulation Outputs for Test System B under Free-Surface Flow Condition 272

10.7 Simulated Outputs for the Five Dual Overflow Test Systems with Various Rainfall Intensities 273

10.8 Percentage Difference between Inflows and Combined Outflows for Surcharge Component Test 274

10.9 Predicted Levels for Test System B under Free-Surface Flow Conditions 274

10.10 Predicted Levels for Various Altered Test Systems under Surcharged Flow Conditions 275

10.11 Heads above Weir Crest with Corresponding Overflows at the Two Cross-Connections for Test System B 276

10.12 Details of Altered Parallel Pipe System and Computed Reverse Flow together with Hand-Calculation Check 277

10.13 Continuity of Flow along the Parallel Pipes for the Double Cross-Connections Test System 278

10.14 Peak Discharges and Ratio of Flows in Parallel Pipes under Steady State Condition 279

10.15 Details of Events for Simulation 280

10.16 Events and the Purpose of Modelling for the Three Parallel Pipe Catchments 281

10.17 Observed and Computed Peak Discharges in Parallel Pipes for Rex Park and Mill Road Catchments 282

10.18 Observed and Computed Peak Discharges in Parallel Pipes for Bothwell Street Catchment 283

10.19 Observed and Predicted Total Runoff Volumes in Parallel Pipes for Rex Park and Mill Road Catchments 284

10.20 Observed and Predicted Total Runoff Volumes in Parallel Pipes for Bothwell Street Catchment 285

10.21 Catchment Sizes and Dry Weather Flows for Three Parallel Parallel Pipe Catchments 285

10.22 Nett Runoff Volumes for Rex Park and Mill Road Parallel Pipe Catchment 286

10.23 Predicted and Nett Runoff Volumes for Bothwell Street Catchment 287

10.24 Combined Runoff Volumes and Ratio of Flows in Parallel Pipes for Bothwell Street Catchment 288

10.25 Combined Flow Volumes and Related Flow Percentage for the Three Catchments 289

10.26 List of Events and the Corresponding Combined Runoff Volume and RUNVOL for the Three Catchments 290

10.27 Comparison of Runoff Coefficient (RC) and Depression Storage (DEPSTOG) Values between those Derived Using Observed Data and the Parallel Pipe Model Results 291

10.28 Comparison of Observed and Predicted Peak Levels in Parallel Pipes for Three Catchments 291

LIST OF FIGURES

		Page
2.1	The Hydrological Cycle in Pictorial Form	22
2.2	The Hydrological Cycle in Systems Notation	22
2.3	The Urban Hydrological Cycle	23
2.4	The Effects of Urbanisation on Hydrological Processes	23
2.5	Examples of Smoothed Rainfall Hyetographs for Dunfermline	24
2.6	The Equations of Continuity (A) and of Motion (B) for Unsteady Gradually Varied Open Channel Flow	25
3.1	Representation of the Continuity Equation	51
3.2	Characteristics of Space / Time Grid	51
3.3	Rectangular Grid for the Characteristic Method	52
3.4	Fixed Rectangular Space / Time Grid	52
3.5	Finite Difference Grid for Kinematic Methods	53
3.6	Error Curves Showing Recommended Restrictions on ξ and $1/\omega r$	54
4.1	Preissmann Slot -- Circular Pipe with Fictitious Slot	70
4.2	Selection of the Appropriate Method in WASSP Package	71
4.3	Sewered Sub-Area Model in Wallingford Procedure	72

4.4	Conceptual Flowchart for DUCTS Sewerage Simulation Model	73
5.1	Location of Dunfermline in Fife Region	86
5.2	Lyneburn Catchment at Liggar's Bridge	87
5.3	Locations of the Five Main Sewers in Dunfermline	88
5.4	Main and Subsidiary Sewers in Lyneburn Study Catchment	89
5.5	Locations of Sub-Areas for Detailed Overland Survey and the 'Critical' Sewers	90
5.6	Sewered Sub-Areas	91
5.7	Recorded Water Levels at Time 16:26 due to Total Rainfall of 32.6mm on 17 June 1986	92
5.8	Global and Sub-Catchment Boundaries	93
5.9	Detail Layout of Parallel Sewers	94
5.10	Diagrammatic Sketch of the Two Typical Types of Cross-Connection Overflows	95
6.1	Suggested Numbers of Raingauge for Lyneburn Catchment	131
6.2	Dynamic Calibration Results for Lambrecht 0.1mm Tipping-Bucket Raingauges	132
6.3	Installation of Sewer Flow Survey Unit	133
6.4	Flow Logger Sensor Head	134
6.5	The Parameters of the Circular Cross-Section	134
6.6	Static Depth Check for DET Sewer Flow Survey Logger	135

6.7 Percentage Errors in Depth and Velocity for DET Flow Survey Loggers 136

6.8 Locations of Data Capturing Equipment 137

6.9 Typical Rainfall Data Printouts 138

6.10 Graphical Representation of Recorded Sewer Flow Data 139

6.11 Data Handling Procedure and the Associated Computer Software 140

6.12 Typical Scattergraph Plot (Bothwell Street -- Foul) 141

6.13 Roughness (Ks) Determination for Scotland Drive 142

6.14 Roughness (Ks) Determination for Rex Park (Foul) 143

6.15 Roughness (Ks) Determination for Rex Park (Storm) 144

6.16 Dry Weather Flows at Rex Park 145

6.17 Dry Weather Flows at Mill Road 145

6.18 Dry Weather Flows at Bothwell Street Chamber 146

6.19 Runoff Volumes against Rainfalls for the Lyneburn Catchment at Various Locations 147

6.20 Water Depths in Manholes at Cross-Connection 5 due to a Storm with 32.6mm Rainfall on 17 June 1986 148

7.1 Scotland Drive Sub-Catchment 166

7.2 Garvock Bank Sub-Catchment 167

7.3 Diagrammatic Sketch of Full and Simplified Models for Scotland Drive Sub-Catchment 168

7.4 Comparison Among Simplified and Full Simulation Model with the Observed Flow at the Outfall 169

7.5 Comparison between WASSP and DUCTS Simulation Outputs at Scotland Drive Sub-Catchment Outfall 170

7.6 Comparison between WASSP and DUCTS Simulation Outputs at Garvock Bank Sub-Catchment Outfall 171

7.7 Comparison between Observed and WASSP Computed Flows at the Outfall of Calais Burn 172

7.8 Comparison between Observed and WASSP Computed Flows at the Outfall of Rex Park 173

7.9 Comparison between Observed and WASSP Computed Flows at the Outfall of Mill Road 174

7.10 Comparison between Observed and WASSP Computed Flows at the Outfall of Bothwell Street Chamber 175

7.11 Comparison between Observed and WASSP and DUCTS Computed Flows at the Outfall of Calais Burn 176

7.12 Comparison between Observed and WASSP and DUCTS Computed Flows at the Outfall of Mill Road 177

7.13 Comparison between Observed and WASSP and DUCTS Computed Flows at the Outfall of Mill Road 178

7.14 Comparison between Observed and WASSP and DUCTS Computed Flows at the Outfall of Central Park 179

7.15 Comparison between Observed and WASSP and DUCTS Computed Flows at the Outfall of Bothwell Street Chamber 180

7.16 Peak Discharges of WASSP against Observed Values 181

7.17 Peak Discharges of DUCTS and WASSP against Observed Data 181

8.1 Diagrammatic Sketch of Overflow at Cross-Connection 197

8.2 Classification of Side-Weir Overflow 198

8.3 Possible Overflow Behaviour in Typical Cross-Connections 199

8.4 Relationship between Proportional Discharge and Proportional Depth 200

9.1 Flow Chart Showing the Enhancements Required for Each Major Subroutine in DUPPERS 227

9.2 Downstream Manhole Numbers for Single and Parallel Pipe Systems 228

9.3 Branch and Pipe Numbering System for Parallel Pipes and Cross-Connections 229

9.4 The Structure of Parallel Pipe Model DUPPERS 230

9.5 Arrays Set-up for Storing Critical Locations in Cross-Connections 231

9.6 Flow Pattern for a Typical Cross-Connection 232

9.7 Possible Flow Behaviours in the Lyneburn Cross-Connection Overflows 233

9.8 Possible Surcharged Sub-Systems along the Parallel Pipes and Cross-Connections 234

10.1 Schematic Sketch of 'Idealised' Test Systems 292

10.2 Schematic Sketch of Cross-Connection in Test Systems A and B 292

10.3 The Three Artifical Test Rainfall Patterns 293

10.4 Proposed Surcharged Sub-System Set-ups for Component Testing 294

10.5 Overflow against Weir Head for Weir Type 1 295

10.6 Cross Sectional Sketch Showing Balance of Heads in Cross-Connection Manholes 295

10.7 Simulation of Flows for Parallel Pipes for Test Systems B1, B3 and B5 using Test Rainfall Type 1 296

10.8 Simulation of Flows for Parallel Pipes for Test Systems B1, B3 and B5 using Test Rainfall Type 2 297

10.9 Simulation of Flows for Parallel Pipes for Test Systems B1, B3 and B5 using Test Rainfall Type 3 298

10.10 Setting Up a Sewer Flow Simulation Model 299

10.11 Response of Flows in Parallel Pipes at Bothwell Street under Various Uniform Rainfalls 300

10.12 Peak Discharges against Uniform Rainfalls for Parallel Pipes at Bothwell Street 301

10.13 Ratio of Flow to the Combined Flow against Uniform Rainfall 302

10.14 Comparison of Observed and Predicted (DUPPERS) Flows for Parallel Pipes at the outfalls of Rex Park Catchment 303

10.15 Comparison of Observed and Predicted (DUPPERS) Flows for Parallel Pipes at the outfalls of Mill Road Catchment 304

10.16 Comparison of Observed and Predicted (DUPPERS) Flows for Parallel Pipes at the outfalls of Bothwell St Catchment 305

10.17 Verification of Model by Comparing the Observed data and DUPPERS Simulated Outputs for Parallel Pipes at Rex Park 306

10.18 Verification of Model by Comparing the Observed data and DUPPERS Simulated Outputs for Parallel Pipes at Mill Road 307

10.19 Verification of Model by Comparing the Observed data and DUPPERS Simulated Outputs for Parallel Pipes at Bothwell Street Chamber 308

10.20 Verification of Model by Comparing the Observed data and DUPPERS Simulated Outputs for Parallel Pipes at Bothwell Street Chamber 309

10.21 Computed against Observed Peak Discharges for Parallel Pipes at Rex Park 310

10.22 Computed against Observed Peak Discharges for Parallel Pipes at Mill Road 311

10.23 Computed against Observed Peak Discharges for Parallel Pipes at Bothwell Street 312

10.24 Computed Peak Discharges against Rainfall Depths for Parallel Pipes at Rex Park 313

10.25 Computed Peak Discharges against Rainfall Depths for Parallel Pipes at Mill Road 314

10.26 Computed Peak Discharges against Rainfall Depths for Parallel Pipes at Bothwell Street 315

10.27 Computed Peak Discharges against Peak Rainfall Intensities for Parallel Pipes at Rex Park. 316

10.28 Computed Peak Discharges against Peak Rainfall Intensities for Parallel Pipes at Mill Road 317

10.29 Computed Peak Discharges against Peak Rainfall Intensities for Parallel Pipes at Bothwell Street 318

10.30 Computed Total Runoff Volumes against Total Observed Runoff Volumes for Parallel Pipes at Rex Park 319

10.31 Computed Total Runoff Volumes against Total Observed Runoff Volumes for Parallel Pipes at Mill Road 320

10.32 Computed Total Runoff Volumes against Total Observed Runoff Volumes for Parallel Pipes at Bothwell Street 321

10.33 Computed Runoff Volumes against Rainfall Depths for Parallel Pipes and the Combined at Bothwell Street 322

10.34 Runoff Volume (RUNVOL) against Rainfall Depth (P) for Rex Park Catchment 323

10.35 Runoff Volume (RUNVOL) against Rainfall Depth (P) for Mill Road Catchment 324

10.36 Runoff Volume (RUNVOL) against Rainfall Depth (P) for Bothwell Street Catchment 325

10.37 Runoff Volume (RUNVOL) against Rainfall Depth (P) for all Three Locations 326

10.38 Comparison of Observed and Predicted Levels for Parallel Pipes at the outfalls of Rex Park 327

10.39 Comparison of Observed and Predicted Levels for Parallel Pipes at the outfalls of Mill Road 328

10.40 Comparison of Observed and Predicted Levels for Parallel Pipes at the outfalls of Bothwell Street 329

10.41 Predicted against Observed Peak Levels for Parallel Pipes Based on the Results of the Three Level Hydrographs 330

LIST OF PLATES

		Page
5.1	Dry Weather Flow in the Lyne Burn at Rex Park in March 1985.	96
5.2	High Flow in the Lyne Burn at Rex Park in May 1985.	96
5.3	Storm Outfall for a Housing Estate allows Combined Discharge into the Burn.	97
5.4	Parallel Pipes and Cross-Connection No. 4 at Rex Park.	97
5.5	Surface Flooding occurs Frequently at Rex Park due to the Shallow Manhole Depth and Mild Catchment Slope.	98
5.6	An Overflow Occurred in Bothwell Street Chamber during a Storm on 1 April 1985.	98
6.1	Complete Set-Up of Dynamic Calibration for the Lambrecht 0.1mm Tipping-Bucket Raingauge.	149
6.2	A DET Flow Survey Logger Being Hung just Underneath the Manhole Cover to Avoid Surcharge.	149
6.3	An Ultrasonic Sensor (Mouse) Fixed on a Steel Band was Installed in the Invert of a Sewer.	150
6.4	Complete Set-Up for DET Logger Free Surface Calibration, Entrance was Tapered to Avoid Turbulence in the Pipe.	150
6.5	The Exit End of the Calibration Channel and the Weighing Tank to Determine Flowrate.	151
6.6	The Raingauge Site at Scotland Drive for the Lambrecht 0.1mm Tipping-Bucket Gauge.	151

6.7 Raingauge Site at Brucefield House for the Lambrecht 0.1mm Tipping-Bucket Gauge. 152

6.8 Raingauge Site at Police Headquarter Roof for the 0.5mm IH Daily Totaliser Gauge. 152

NOTATION

A	- Cross-sectional area
Ae	- Wetted channel cross-sectional area
Ao	- Full-bore cross-sectional area
Ao	- Cross-sectional area of orifice
Ap	- Proportional area
a	- Coefficient
B	- Surface width
Bs	- Width of Preissmann slot
b	- Breadth of channel
C	- Capillary rise (Figures 2.2 and 2.3)
C1,C2,C3	- Muskingum-Cunge coefficient
Cd	- Coefficient of discharge
c	- Convection speed
D	- Hydraulic depth
Dp	- Proportional depth
d	- Diameter
de	- Diameter of bridging pipe
E	- Evaporation
e	- soffit level
F	- Friction factor
F	- Infiltration (Figures 2.2 and 2.3)
f	- Flowrate
fe	- Friction factor of bridging pipe
g	- Acceleration due to gravity
H	- Head over a weir
Hav	- Average head above weir
Ho	- Head above orifice
Hω	- Head above weir
h	- Water level
I	- Inflow from a reach
i	- Excess rainfall intensity in mm/hr (Chapter 2)
j	- Time label
K	- Muskingum method parameter
Kr	- Storage constant
Ks	- Pipe roughness
L	- Pipe or reach length
Le	- Length of bridging pipe

$L\omega$ - Length of weir
M - Moisture transfer (Figures 2.2 and 2.3)
O - Outflow from a reach
P - Plan area
P - Wetted channel perimeter (Chapter 3)
P - Precipitation (Figures 2.2 and 2.3)
p - Weir height (Chapter 8)
Q - Discharge
Qfb - Full-bore discharge
Qin - Inflow
Qorif - Discharge through orifice
Qp - Peak discharge
Qp - Proportional discharge (Chapter 8)
Qw - Discharge over weir
Qb,Qg,Qi,Qs - Throughflow (Figures 2.2 and 2.3)
q - Discharge per unit area (Chapter 2)
q - Lateral inflow
R - Hydraulic radius
R - Deep percolation (Figures 2.2 and 2.3)
Rp - Proportional radius
r - Ratio of time increment to space increment
S - Storage
Sf - Friction slope
So - Bed slope
s - Bottom slope
t - Time
V - Volume of water in manhole above outgoing pipe soffit
Vav - Average velocity
v - Velocity
w - Weir length
x - Distance
x - Muskingum weighting parameter (Chapter 3)
y - Depth

ν - Kinematic viscosity
κ - Headloss coefficient
Θ - Finite difference weighting coefficient
μ - Diffusion parameter
α - Attenuation parameter

ω - Kinematic wave speed

θ - Angle sustained from the centre of circle

λ - ratio of time base to time increment

ϕ - Relative roughness value

ξ - Muskingum method parameter

β - Weir equation constant

βe - Constant

Δd - Depth increment

ΔQ - Side spill discharge

ΔV - Velocity increment

Δx - Space increments

Δt - Time increment

AREAC - Catchment area (ha)

CWI - Catchment wetness index

DEPSTOG - Depth of depression storage (mm)

P - Rainfall depth (mm)

PAPG - Paved area per gulley

PIMP - Percentage of catchment covered by impervious surfaces intended to drain to the storm sewer

PR - Percentage runoff from total catchment area

PRpav - Percentage runoff from paved area

PRperv - Percentage runoff from pervious area

RC - Runoff coefficient

RUNVOL - Runoff volume

SLOPE - Subcatchment slope

SOIL - Soil index

SPR - Standard percentage runoff

UCWI - Urban catchment wetness index

URBAN - Urban catchment size

Suffix

i, j, k - Node label

1, 2 - Node label

GLOSSARY OF TERMS

Combined System -- Sewerage system in which both foul and stormwater flows are conveyed in the same pipe.

Cross-Connection -- A linking pipe connecting foul and stormwater sewers in a parallel pipe system. Flow is possible in either direction.

Depression Storage -- The depth of water retained on the ground surface in puddles or other depressions.

Free Surface Flow -- Flow conditions which include a water surface subject to atmospheric pressure.

Model Calibration -- A simulation process used to check the performance and to identify errors of a model by comparison with observed data. Alterations and/or adjustment may often be required to bring the model close to the observed data.

Model Verification -- A simulation process to justify the final performance of a calibrated computer model by comparing with observed data which is not the same as those in the calibration process.

Parallel Pipe System -- A separate pipe system with both foul and stormwater sewers are laid in close proximity.

Percentage Runoff -- The percentage of rainfall volume falling on a specified catchment area which enters the stormwater drainage system. The catchment area may be composed of impermeable and/or permeable areas.

Rainfall Volume -- The depth of observed rainfall multiplied by the total contributing catchment area.

Runoff Coefficient -- A factor accounting for additional losses after the depression storage has been filled, to make the rainfall volume equal the runoff volume plus total losses for a specified surface type.

Runoff Volume -- The total volume of flows (foul and storm) passing the outfall point due to a rainfall event.

Separate System -- Sewerage system in which foul and stormwater flows are conveyed in separate pipes.

Simulation Model -- Computer model, representing a sewerage system, assembled by means of some mathematical functions and equations, and used to predict flow and/or level information at desired locations within the system.

Single Lumped Pipe Model -- A simulation model combining parallel pipes to form a system with a single equivalent pipe.

Sub-Catchment -- An contributing area draining to a single pipe length.

Surcharged (Pressurised) Flow -- Flow conditions in which the water level at manholes is higher than the pipe soffits.

CHAPTER 1
INTRODUCTION

1.1 GENERAL INTRODUCTION

Urban storm drainage systems are complicated networks designed to convey excessive rainwater away from the urbanised areas. Storm sewer systems are underground networks of pipes in which branch sewers converge to form a trunk sewer which then discharges to an outfall either to a suitable watercourse or another part of the drainage system. Such networks are often described as tree-like dendritic systems without loops or diverging pipes (Walters 1982). Somtimes duplicate pipes besides trunk sewers form parallel pipe systems which may be with or without loops. In the past, culverting of streams was found to be sufficiently adequate to convey waste. With the increasing population and urbanisation, however, the social and environmental advantage of purposely built drains was recognised resulting in the increased use of underground pipe networks.

Sewers conveying surface water are normally designed to operate unsurcharged and under gravity with flows calculated by such as the Rational Method (HMSO 1976). In such methods the flow at a point in the network is dependent on the nature of the network upstream of that point. With the increase of flow due to urbanisation, complex flow hehaviour is possible which can normally be classified into two distinct modes : open channel (free surface) flow and surcharged (pressurised) flow, in which the flow rates vary with time.

Free surface flow obeys the conventional hydraulic behaviour of free surface systems at atmospheric pressure. Analytical techniques used in this flow condition are similar to flood wave analysis in river channels, canals and reservoirs. A flood wave may be defined broadly as a temporal and spatially propagated change in water surface, discharge or velocity. Propagation of flood waves is described mathematically by the equations of free surface unsteady flow.

In the surcharged condition flow will take place under a pressure created by the hydraulic head established in manholes. Surcharging occurs whenever the flow is greater than the pipe capacity, or when downstream conditions restrict the flow by means of a backwater effect. In most sewer systems, surcharging is often unavoidable and frequent surcharging is implicitly acceptable so long as it does not exceed the ground level (Colyer 1981).

Storm drainage networks may be broadly classified as either 'combined' or 'separate'. Combined systems are built to convey in a single pipe both storm runoff and domestic foul sewage. Normally storm water flows generated by rainstorms are very much greater than foul sewage flows and it is often impractical to carry such large flows to a sewage treatment works. Most combined systems incorporate storm water overflows which spill a proportion of their flow to a watercourse during storm conditions. Separate sewer systems, however, consist of two networks, one for foul sewage and the other for storm runoff. The storm runoff in these systems is generally discharged without treatment to convenient watercourses.

There are also problems associated with overflows in urban drainage networks which normally occur in the densely urbanised areas of larger towns and cities. Many problems can be traced back to the early nineteenth century (Stanbridge 1976). In the past, surface drains were laid along principal streets, discharging to nearby streams or rivers. Gradually domestic outlets were connected (by mistake or for convenience) to the separate system and by the middle of the nineteenth century large volumes of untreated domestic and industrial waste were being discharged to watercourses. With current legislation, such discharges are only allowed for overflow under storm conditions. Many old sewerage systems, therefore, require to undergo rehabilitation process in order to meet the legislation and to upgrade the performance of the systems.

Water authorities in the UK are continually upgrading drainage networks to improve their hydraulic, pollution and structural performance. Three possible options are generally available for a

planned rehabilitation policy (Green & Drinkwater 1985) :

(i) RENEWAL
Recontruction of the existing sewers to accommodate the original design flow ;

(ii) REPLACEMENT
Reconstruction of existing part or whole system to accommodate both the existing and future design flows ;

(iii) RENOVATION
To upgrade existing sewers, which may have sufficient capacity but have deteriorated, by lining materials installed on the inside wall of the pipe.

With the launch of the Sewerage Rehabilitation Manual in April 1984 (WAA/WRc 1984), guidelines and appropriate procedures were set out to assist planning and design engineers responsible for foul and stormwater drainage to achieve the most cost effective solutions for the rehabilitation of sewer systems in the UK. The Manual also sets out strategies which are based on the fact that investigations should be directed to those critical sewers where most of the expenditure will be concentrated. Among the three options above, sewer renovation is now generally available at considerably lower cost than the traditional renewal and replacement or reinforcement solutions. The Manual also stresses that cost effective rehabilitation methods retain as much as practicable of the existing sewer network, optimise hydraulic performance and maximise the opportunities for future renovation. Over the last decade, analytical tools such as simulation models have been developed to cater for design of urban drainage systems and to identify the critical sewers in a system.

These advanced analysis tools for urban drainage networks have revealed a significant demand for reliable field data primarily relating to rainfall and runoff but sometimes also on the other components of the flow process and more recently, water quality parameters. Resistance to the acceptance of analysis based on mathmatical models of existing sewer systems has also resulted from a general lack of confidence in the simulation outputs. Traditional methods such as the TRRL Hydrograph Method did not simulate

surcharged conditions and it was not possible to represent realistically an existing overloaded sewer system. It was often the case that theoretical models were constructed for a particular drainage network limiting their general use for other systems. The recent introduction of packages such as the Wallingford Procedure for the design and analysis of urban storm drainage, in particular the WASSP-SIM program (HRS 1981), has largely been accepted by the UK water authorities as the standard procedure. However, this is very often used as a 'black box' tool and the mis-interpretation of simulation outputs is common. The old adage, 'rubbish in leads to rubbish out' applies equally to any form of mathematical modelling.

1.2 RESEARCH AIMS AND OBJECTIVES

The principal objective of the research project was to construct a mathematical model to simulate a section of parallel sewers with cross-connecting pipes. Despite many available computer models in the UK and worldwide, they are applicable either to dendritic systems or are tailored to their own study networks. Most are too simplistic for this application. Furthermore, the complicated flow phenomena including overflows with weirs, reverse flows and head balance between the twin pipes via the connections preclude the adoption of commercial simulation models. A need therefore existed to build a theoretical simulation model to predict flows in the parallel pipes separately and the flow behaviour in the cross-connections.

The simulation program has been developed using existing above-ground models. Flow routing methods in sewers have been developed from the full Saint-Venant equations. In simulating flows in the parallel pipes and cross-connections under surcharged conditions, it was particularly important to ensure the surcharged sub-systems formed appropriately to allow flow to be routed through the sub-systems.

Different overflow structures and their hydraulic behaviour together with the theoretical equations are all reviewed and an overflow type was selected which was similar to that in the study system. The overflow weirs, either single or double-sided, have been incorporated in the simulation model.

The new parallel pipe model is an enhancement of the existing simulation model DUCTS which has been developed in the Institute for use only on dendritic systems. The general performance of this model, which had not been previously calibrated and verified, was investigated by constructing an combined equivalent pipe model for the Lyneburn parallel pipe system in Dunfermline. The Lyneburn catchment has an area of some 675 ha and a population of approximately 35,000. This exercise resulted in a clarification of contributing areas of the study catchment, and also allowed verification of the catchment data for the parallel pipe model. The performance of the Sewered Sub-Area (SSA) model was investigated by applying it to two sewered sub-catchments. The SSA was applied and utilised to a great extent to simplify many self-contained contributing sub-areas.

Field data capture is vital in any drainage system simulation. Rainfall and sewer flow data were required in this study with all equipment calibrated before use for data collection. The flow data loggers used required substantial calibration checks in order to ensure satisfactory performance and to gain confidence in the data captured. Sufficient events had to be monitored for both rainfall and the corresponding sewer flow records at different locations. A sequential data logging procedure was adopted due to the limited number of flow loggers available.

A more general overall object of the research project was to provide additional information in the field of urban drainage system simulation, to assist in the understanding of sewer flow routing problems, particularly those encountered in parallel pipes with the presence of cross-connections, and to indicate the usefulness and efficiency of the available numerical modelling

techniques. A model of the Lyneburn parallel pipe system was constructed primarily to indicate the benefits and difficulties associated with mathematical modelling of the twin pipes, stormwater overflows and flow behaviour at cross-connections.

1.3 RESEARCH PRESENTATION

A review of urban drainage runoff modelling and the runoff sub-models is provided in Chapter 2. A mathematical representation of the sub-models, which can be classified into the above-ground phase including the rainfall and overland runoff and the below-ground sewer flow phase, has also been presented.

Insight into the different flow routing schemes and their solution techniques has been reported in Chapter 3. Development of the Muskingum-Cunge method is also reviewed together with the application of free surface flow routing in pipes and the choice of the routing parameters.

Important worldwide and UK urban drainage simulation models are selected and reviewed in Chapter 4. These models range from simple simulation techniques unable to deal with surcharged flow, to the sophisticated methods such as Preissmann slot method with the capability of modelling many hydraulic structures in networks.

A description of the Lyneburn study catchment and parallel pipe system is reported in Chapter 5. The flow behaviour problems in parallel pipes and cross-connections are also discussed. Equipment used for data collection and the calibration details are included in Chapter 6. The data capture procedure and the method of checking observed data are also described in this chapter.

The necessity of catchment simplification and the process of simplifying the Lyneburn system are reported in Chapter 7. Verification of the SSA model and catchment data and the performance of the single lumped pipe model is also discussed and presented.

A review of the overflow structures available in urban drainage systems, their development and hydraulic representation is summarised in Chapter 8. Mathematical equations for storm sewer overflows are also presented with their application to the Lyneburn parallel pipe system. The level calculation procedure for free surface flow is enclosed in this chapter. A description of the enhanced computer program is detailed in Chapter 9 for those subroutines which were developed and enhanced.

A critical appraisal of the newly developed components in the parallel pipe model and the test results are presented in Chapter 10, based on test systems with artificial rainfall. The tests on the parallel pipe system, used both artifical and observed events, and the simulation outputs are also supplied in this chapter. Finally Chapter 10 includes an analysis of computed outputs and comparisons with the observed data.

An overview of the model is presented in Chapter 11 while Chapter 12 covers general conclusions and suggestions for further work.

Tables, Figures and Plates are provided at the end of each Chapter. Details of the computer program DUPPERS and solution techniques are provided in Appendix A. Lists of Tables, Figures, Plates and Terms of Glossary are all included at the beginning of the thesis.

CHAPTER 2
DEVELOPMENT OF URBAN DRAINAGE RUNOFF MODELLING IN THE UK

2.1 INTRODUCTION

Hydrology may be defined as the physical science of the waters of the Earth, their occurrence, circulation and distribution, their chemical and physical properties, and their reaction with the environment, including their relation to living things (UNESCO 1979). This is expressed in the hydrological cycle, which illustrates the multifarious paths by which water precipitated on to the land surface finds its way to the oceans, with evaporation providing the supply of moisture for the renewal of the process. The hydrological cycle is commonly presented in pictorial form, of which Figure 2.1, adapted from Todd (1959), provides a typical example. Although Figure 2.1 is useful in imparting the essential features of a water cycle driven by the excess of incoming over outgoing radiant energy, this representation fails to provide an adequate framework for the study of its component processes. Such a framework can be obtained by adopting the so-called systems notation, in which the paths of water transport link the major sources of moisture storage as presented by Dooge (1973), in Figure 2.2. The subsystem, which is bounded by the broken line in Figure 2.2, refers to the land phase of the hydrological cycle. It receives an input of precipitation P, and produces outputs in the form of evaporation E, and stream flow Q.

Perhaps the most obvious definition of urban hydrology would be the study of the hydrological processes occurring within the urban environment. However, further consideration of the hydrological cycle of an urban area, as presented by Hall (1984) in Figure 2.3, soon reveals the inadequacy of this simplistic representation. Natural drainage systems are both altered and supplemented by sewerage and the effects of flooding are mitigated by flood alleviation schemes or storage ponds. Several authors, including Savini and Kammerer (1961), Leopold (1968), Hall (1973) and Cordery (1976), have described the changes in flow regime which occur when an initially rural catchment area is subjected to urbanisation.

The particular aspects of urbanisation which exert the most obvious influence on hydrological processes are the increase in population density and the increase in building density within the urban area. The consequences of such changes are outlined diagrammatically in Figure 2.4 (Hall 1984). Owing to the larger impervious areas, a greater proportion of the incident rainfall appears as runoff than experienced by catchments in the rural state. Furthermore, the laying of storm sewers and the realignment and culverting of natural stream channels which takes place during urbanisation result in water being transmitted to the drainage network more rapidly. This increase in flow velocities directly affects the timing of the runoff hydrograph.

Expanding urbanisation is the cause of Dunfermline's drainage problems. Recent urban development has substantially increased the size of the Burgh, and this growth progressed even more rapidly after the opening of the Forth Road Bridge. The drainage problem of Dunfermline has currently been under review and the report by Ashley and Jefferies (1983) tackles the main problems. The detail of the drainage systems of Dunfermline will be discussed in Chapter 5.

2.2 PRECIPITATION

During the nineteenth century, information on heavy falls of rain in short periods in the British Isles was collected by the British Rainfall Organisation, a group of volunteer observers whose data were collected and published by their founder, G.J. Symons, in an annual publication entitled **British Rainfall**. The British Rainfall Organisation published their first table of heavy rainfalls in short periods in 1888. These data, which were classified as either 'noteworthy' or 'exceptional', may be regarded as one of the first attempts to compile a rainfall depth-duration-frequency (DDF) relationship. With the introduction of autographic rainfall recorders during the 1920s, more reliable data began to be acquired and more statistical analyses permitted a more precise definition

of relative frequency. The relationship between rainfall intensity-duration-frequency (IDF) and DDF since then has been investigated thoroughly by researchers, including Bilham (1935), Norris (1948), Holland (1964, 1967), Rodda (1966), Folland et al (1981) and Colyer (1981).

2.2.1 RAINFALL DEPTH-DURATION-FREQUENCY RELATIONSHIPS

The relationships discussed here are statistical abstractions of point rainfall, i.e. precipitation as observed at a single raingauge. These data are sufficient to allow the following generalisation to be made about the characteristics of storm rainfall:

(1) As storm duration increases, the average rainfall intensity decreases for any given frequency of occurrence; and

(2) As the frequency of occurrence decreases, the average rainfall intensity increases for any given duration.

These generalisations relate to temporal variations in rainfall depth only. In order to evalute the spatial characteristics of storm rainfall, data from networks of raingauges must be studied. From the results of such investigations, a third feature of heavy rainfall has become well established by observation:

(3) The greater the area covered by a storm, the lower the average rainfall intensity compared with the maximum point rainfall intensity recorded within the storm boundaries.

This reduction in areal average rainfall intensity with respect to the maximum intensity observed may be described for individual events by means of a rainfall depth-area-duration (DAD) relationship. However, when attempting to adjust a design (point) depth to allow for spatial distribution, the ratio of the areal average to the point average rainfall intensities corresponding to the same frequency of occurrence is required. This ratio, which is referred to as the areal reduction factor (ARF) (Bell 1976) is

often confused with the rainfall DAD relationship, although both are useful for distinctly different purposes and further details can be found in the Flood Studies Report (NERC 1975) and Bell (1969). In addition, since many design flood estimating methods require a single, representative rainfall input for a complete catchment area, techniques for averaging the records from several raingauges are often required.

Bilham (1935), who used autographic rainfall recorders over a total of 12 sites, analysed the collected data using the rainfall DDF relationship of the form :

$$N = 1.25 \cdot (t / 60) \cdot (R + 0.1)^{3.55} \qquad (2.1)$$

where N is the average number of times a rainfall of depth R (mm) and duration t (min) is equalled or exceeded in 10 years. Equation 2.1 was intended to be applicable for durations between 5 and 120 minutes. Perhaps the major criticism of Bilham's formula is its failure to take account of regional variability in the frequency of heavy rainfall. In the 1960s, Bilham's formula was revised by the Meteorological Office as reported by Holland (1964) and Ashworth & O'Flaherty (1974).

2.2.2 STORM PROFILES

Initially, storm profiles were envelope curves constructed by integrating the rainfall DDF relationship over time. However, with the accumulation of data from autographic raingauges, analysis of selected storm events became feasible.

The rainfall profiles quoted by the Road Research Laboratory (HMSO 1963), a tabulation of which was later provided by Watkins (1966), continued in use until the publication of the Flood Studies Report (NERC 1975). This revision was based upon the analysis of a wider range of storm events having durations up to four rain-days. Each storm was centred on the shortest duration giving at least half the rainfall total, resulting in a set of mean profiles that were symmetrical but varied in amplitude. These profiles were then ranked according to 'percentile peakedness', i.e. the percentage of

occasions when storms were less peaked than a given mean profile. Separate analyses were carried out for summer (May to October) and winter (November to April) storms, and in both cases variations in profiles with storm duration and the return period of the average intensity of rainfall during the storm were found to be relatively insignificant.

Typical examples of storm profiles of different percentiles of peakedness can be found in the Flood Studies Report (NERC 1975) and Meteorological Report (Warrilow 1980). The 50% summer storms of one year return period for the Dunfermline area are shown in Figure 2.5. The data for Figure 2.5 were produced by the computer program SMOOTH-RAIN developed at Dundee Institute of Technology (Ashley & Jefferies 1983).

2.2.3 STORM MOVEMENT

As the sophistication of mathematic models representing the relationship between rainfall and runoff has increased, greater demands have been placed on the meteorologist to provide more comprehensive analytical descriptions of storm rainfall. If the mathematical model is capable of accepting what is known as a distributed input, i.e. different zones of the catchment area being modelled having different storm profiles as opposed to a single 'lumped' profile, the question arises as to whether the speed and direction of movement of any design storm should be taken into account.

Previously, any design storm employed as the input to a flood estimation method was invariably assumed to remain stationary with respect to the catchment area under study. Nevertheless, laboratory studies in which artificial rainfall has been applied to elementary catchments consisting of sloping planes have demonstrated that upstream movement parallel to the main channel reduces, while downstream movement enhances, the peak rate for a stationary storm (Yen & Chow 1969, Folland & Shaw 1979). Unfortunately, as noted by Shearman (1977), there is a dearth of statistical information on the speed and direction of observed

storm movement and the areal extent of storms. However, studies by Felgate & Read (1975), Shearman (1977) and Marshall (1980) have provided some preliminary results using statistical techniques involving the cross-correlation of records from pairs of raingauges in a network.

The results obtained by Shearman have been used in numerical experiments by Sargent (1981, 1982) to investigate the importance of storm movement in the design of sewer networks. The results obtained showed that, although peak runoff rates could be increased by downstream storm movement, the amount of enhancement was negligible, being a maximum of about 1% when storm velocity was equal to flow velocity. In contrast, when the apparent storm duration was less than the time of flow, substantial reductions in peak runoff rates compared with those of stationary storms could be produced with catchments between 2 and 10 km^2 in area. This tendency towards over-estimation caused by storm movement is obviously worthy of further investigation, perhaps using the more sophisticated models of storm sequences such as those described by Sieker (1977) for the Hamburg area and Amorocho & Wu (1977) for cyclonic rainfall in northern California.

2.3 OVERLAND SURFACE FLOW

The urban runoff process may be seen as a two-phase phenomenon, incorporating both above-ground and below-ground phases, although, unfortunately, there is no clearcut interface between the two. However, the above-ground phase is very often taken to include the conversion of the rainfall on an element of catchment into the contribution to runoff at the manhole in the sewer system where the manhole is the collection point for the given element of catchment. This phase includes not only behaviour of the water whilst above ground but also the routing of flows through gully traps and pipe runs to the manhole in the sewer system. The above-ground phase deals with the conversion of the rainfall hyetograph into the inlet

hydrograph of the sewer system proper. The above-ground model is considered as comprising three sections :

(i) depression storage,
(ii) runoff volume, and
(iii) surface routing.

The three components of the above-ground phase may be applied to any number of different surfaces types such as paved surfaces and roofed surfaces. Sometimes the pervious areas affect the rainfall-runoff volume relationship (the second sub-model), but have a negligible effect on the mechanics of the change in shape between input and output (the first and third sub-models).

The overland flow process has been studied by many researchers. Initially such efforts were directed toward laboratory experiments, the objective of which was to understand the hydraulics of the process. Such investigations include those by Izzard (1944, 1946), Yu & McNown (1964), Yen & Chow (1969), Ong (1972), Kidd & Helliwell (1977), and Akan (1985).

2.3.1 DEPRESSION STORAGE

The study drainage systems have been analysed by WASSP-SIM, and the following sections are based on the Wallingford Procedure (HRS 1981). In the depression storage sub-model, a fixed volume of rainfall is removed en bloc from the beginning of the input hyetograph, corresponding to the volume of water retained on the impervious surface by initial wetting or in surface depressions and subsequently added back in. Where the storm is part of a longer event following a significant period of rainfall, the depression storage is taken to be fully utilised.

The depression storage used in the Wallingford Procedure is related to slope using data from British and Swedish catchments as :

$$DEPSTOG = C \cdot SLOPE^{0.48} \qquad (2.2)$$

A typical value of C is 0.71. However, the constant term C in Equation 2.2 can be varied according to the catchment charateristics (Kidd 1978[1] and 1978[2], Falk & Niemczynowicz 1979, Pratt & Henderson 1981). Normally, the storage is assumed to be identical for paved and pervious areas, whereas sloping roofs are taken as having a fixed value of 0.4mm. For the Lyneburn study catchment, DEPSTOG values were developed for all sub-catchments and the global catchment (see Chapter 6).

2.3.2 RUNOFF VOLUME

The runoff volume submodel is applied to the rainfall in order to obtain the correct volume of runoff. As a first approximation, the runoff is assumed to be 100% from impervious surfaces and zero from pervious surfaces. Departure from this assumption is then modelled by applying a constant correction factor to the ordinates of the rainfall hyetograph over the paved surfaces. The nature of this departure is a complex function of a large number of storm and catchment variables and lends itself better to a statistical approach than a deterministic one. Such statistical analyses have previously been done by the Institute of Hydrology (Stoneham & Kidd 1977, Kidd & Lowing 1979).

The runoff volume is expressed as a percentage (PR) of storm rainfall and its variability with storm and catchment characteristics is expressed by regression equation as :

$$PR = 0.829\ PIMP + 25.0\ SOIL + 0.078\ UCWI - 20.7 \qquad (2.3)$$

The PR value obtained by Equation 2.3 is then distributed to the three surface types : pervious, paved and roofed. If PR is less than 70% of PIMP, it is assumed that the pervious areas do not contribute and that all the runoff arises from the impervious areas. However, if PR is greater than 0.7 times the PIMP, the excess runoff is assumed to arise from both the pervious and

impervious (paved and roofed) areas in the ratio of 0.3 to 1.0. Hence :

$$PRpav = 70 + \frac{0.3\ (PR - 0.7\ PIMP)}{1 - (0.7\ PIMP\ /\ 100)} \qquad (2.4)$$

$$PRperv = \frac{PR - 0.7\ PIMP}{1 - (0.7\ PIMP\ /\ 100)} \qquad (2.5)$$

These equations produce values of PRperv which vary significantly with PIMP and are zero (since negative values have no meaning) over a wide range of catchment properties. Equations 2.4 and 2.5 have recently been revised (Orman 1985) and the new relationship produce markedly lower values of percentage runoff from pervious areas. The more recent simulation program WALLRUS (HRS 1987), an upgrade version of WASSP, has a better estimation of the PR value for a catchment by allowing users to input their catchment overland data and is better in dealing with parameters such as DEPSTOG and UCWI.

Percentage runoff values were also derived for the Lyneburn study catchment and the sub-catchments based on the observed data (see Chapter 6).

2.3.3 SURFACE ROUTING

The surface routing submodel takes the adjusted rainfall hyetograph and routes it over the particular surface to give the runoff or inlet hydrograph to the sewer system. This is achieved by a lumped parameter approach using the net rainfall as input to a nonlinear reservoir, given by :

$$\text{Continuity} \qquad \frac{dS}{dt} = i - q \qquad (2.6)$$

$$\text{Dynamic} \qquad S = Kr \cdot q^{m} \qquad (2.7)$$

The above equations may be derived from the St. Venant equations (Akan & Yen 1981) applied to the overland flow phenomenon. Equation 2.7 is derived by ignoring the dynamic wave and diffusion wave terms in the St. Venant dynamic equation (in effect, taking steady

uniform flow condition). This is the kinematic wave approximation, which has been shown (Muzik 1974) to be reasonable where lateral inflow predominates (Woolhiser & Liggett 1967). The kinematic wave assumption applies satisfactorily for both overland and channel flow.

A fixed set of K values in Equation 2.7 corresponding to three different classes of slope and three different classes of AREA (per gully) are used in the Wallingford Procedure routing calculation. This yields a standard set of nine inlet hydrographs (plus one for all pitched roofs) which can be applied to the respective surface areas in each subcatchment.

2.4 SEWER FLOW ROUTING

The inlet hydrographs are combined and routed through the sewer system to the outfall. Pipe-routing is achieved by kinematic routing, in which each ordinate of a hydrograph is offset by a time corresponding to the velocity of flow for that discharge. A more complex technique, using the method of characteristics solution to the St. Venant equations (de Saint-Venant 1871) may also be used (Shepherd 1979) but WASSP uses the simpler solution. However, the continuity equation is also valid and common for all the different types and therefore it is in the equation of motion where differences appear. The classification based on the routing method for open channel flow may be as follows :

(i) Unsteady dynamic wave
(ii) Quasi-steady dynamic wave
(iii) Diffusive wave
(iv) Kinematic wave

The four classes above are listed in order of increasing number of approximations to the original complete equation of motion in Figure 2.6 and with the unsteady dynamic wave being the most

complete. The theoretical basis and the parameters involved in those equations in Figure 2.6 have been investigated by many researchers such as Yen (1973), Sjoberg (1976) and Jensen (1981) and fuller details has been presented in Chapter 3.

2.4.1 FREE-SURFACE FLOW ROUTING

The relationship between discharge, Q, and depth, y, for free-surface steady flow in a sewer is defined using the Colebrook-White equation with the normal depth relationship given the form :

$$Q^2 = 32\ g\ A^2\ R\ s\ \log^2 \left[\frac{Ks}{14800\ R} + \frac{2.51\ \nu}{R\ \sqrt{(128\ g\ R\ s)}} \right] \qquad (2.8)$$

For unsteady flow in a sewer, a proper simulation should be based on a solution of the full Saint-Venant equations. However, a numerical solution of the equations for free-surface flow is complicated by the hydraulic problems of transitions between sub- and super-critical flow and moving hydraulic jumps, conditions at junctions, difficulties with small depth flows and the consequent computational problems of large computer storage and core time requirements. It is necessary to adopt a simpler form of the equations. After the assumptions that reverse free-surface flow rarely occurs, backwater effects in a pipe are minimal and head losses at manholes are kept to a minimum, the simplified Saint-Venant equation (Price 1981) can be expressed as :

$$\frac{\partial Q}{\partial t} + \omega\ (Q)\ \frac{\partial Q}{\partial x} = \omega\ \frac{\partial}{\partial x} \left[a\ (Q)\ \frac{\partial Q}{\partial x} \right] \qquad (2.9)$$

$$\text{where} \quad \omega = \frac{Q}{B} \left[\frac{B}{A} + \frac{1}{F} \frac{dF}{dy} \right] \qquad (2.10)$$

$$\text{and} \quad a = \frac{Q}{2\ s\ B\ \omega} \qquad (2.11)$$

with B, A, F and dF/dy being evaluated for y determined in terms of

Q from the following :

$$Q = A\ F\ s^{1/2} \qquad (2.12)$$

For the numerical computation, Equation 2.9 is rewritten in the form of :

$$\frac{\partial}{\partial t} \int^{Q} \frac{dQ}{\omega} + \frac{\partial Q}{\partial x} + \frac{1}{\omega} \frac{\partial}{\partial t} \left[a \frac{\partial Q}{\partial x} \right] = 0 \qquad (2.13)$$

A centralised 4-point finite difference scheme is then used on this equation. The advantage of using Equation 2.13 rather than 2.9 is that the former equation does not require a downstream boundary condition for its solution and the finite difference algorithm is directionally explicit.

For practical application in sewers a linearised form of Equation 2.13 is adequate. In this case the finite difference scheme corresponds to the Muskingum-Cunge routing method (Cunge 1969). Given the time increment, the space increment for the finite difference scheme is chosen to minimise the errors in convection speed and attenuation of peak discharge between the solution of the linearised form of Equation 2.9 and of the linearised finite difference scheme based on Equation 2.13 (Price 1980).

2.4.2 PRESSURISED PIPE ROUTING

An advantage of the Muskingum-Cunge routing technique is that it may be applied to each pipe separately and in sequence down a branch of the network, and manhole storage may be ignored. However for a group of pipes which are surcharged, the water levels in the manholes and discharges in the pipes have to be evaluated simultaneously because of the instantaneous response of the water levels to any changes of inflow (Bettess & Price 1978). In a sewer network this is achieved by identifying those pipes which, at a given time, are in a connected surcharged group. The flow of water

in a pressurised pipe may be described by the equation:

$$\frac{L_j}{gA_j}\frac{dQ_j}{dt} + h_k - h_j + \left\{\frac{L_j}{F_j^2} + \kappa_j\right\}\frac{Q_{jk}}{2g\,A_j^2}\left|Q_{jk}\right| = 0 \qquad (2.14)$$

$$\text{where } F_j^2 = 4g\,d_j\,\log_{10}^2\left\{\frac{ks}{3700\,d_j} + \frac{2.51\,\nu}{d_j\sqrt{(2g\,d_j\,s_j)}}\right\} \qquad (2.15)$$

The headloss coefficient reflects entrance, bend and exit losses to the pipe, but for convenience is notionally regarded as being at the upstream manhole. The water level in a manhole changes according to the inflow and outflow:

$$\frac{dV_j}{dt} - \sum_i Q_{ij} + Q_{jk} - Qin = 0 \qquad (2.16)$$

It is also assumed that the volume in the jth manhole is given by:

$$Vj = Pj\,(\,hj - ej) \qquad (2.17)$$

The finite difference equations are solved for a succession of small time increments such as one second, the number depending on the time increment used in calculating free surface flow in the non-surcharged pipes (typically 10 seconds). At the end of a longer time increment the pipes are examined to see if the group of connected surcharged pipes should be increased or decreased. The criteria to decide whether a pipe is surcharged or not includes tests on whether the discharge is greater than the pipe-full discharge and whether there is back surcharge from the manhole downstream (Price 1981). Surface flooding is simulated by redefining the equation for Vj in terms of hj to include the large change in plan area that may occur when hj is above ground level.

2.5 THE FRAMEWORK OF DUPPERS

The parallel pipe model DUPPERS is based upon the in-house

simulation program DUCTS. A fuller descriptions of DUCTS is included in Chapter 4. However, the basic structure of the DUCTS model and its operating procedures are as outlined above.

The enhanced model DUPPERS consists of several sub-models as in DUCTS. Those sub-models have been described in Sections 2.2 to 2.4 in this Chapter namely the rainfall, overland runoff and below-ground sewer flow routing sub-models.

The rainfall sub-model (Section 2.2) reads the details of observed rain-storm and the size of catchment. The determination of areal reduction factor and the smoothing of the point rainfall profiles are also encountered in the sub-model.

The hydrological processes of overland urban storm runoff are included in the above-ground flow sub-model (Section 2.3). Overland flow is generated from the catchment as the storm continues after the depression storage is satisfied. The determination of percentage runoff and urban catchment wetness index are computed in this sub-model according to the supplied catchment size and percentage of paved areas. Distribution of runoff volume within the catchment is also included in this sub-model by determining the 10 standard runoff hydrographs.

Flow routing in sewers is performed in the pipe flow routing sub-model (Section 2.4). Flows under both free surface and pressurised conditions are dealt with separately. Under free surface, routing of flow is performed for each pipe individually. For surcharged flow condition, a series of pipes comprising a surcharged sub-system is identified and solved simultaneously. A significant enhancement in DUPPERS was the procedure to identify surcharged sub-systems in the parallel pipe network.

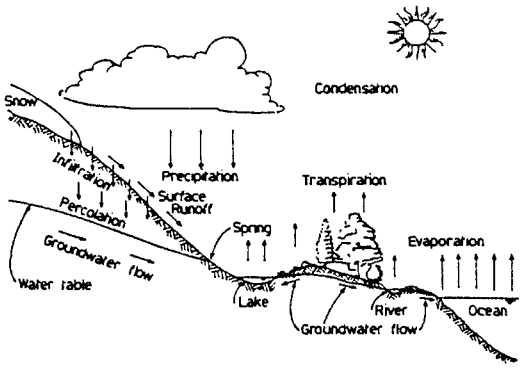


FIGURE 2.1 THE HYDROLOGICAL CYCLE IN PICTORIAL FORM
(Todd 1959)

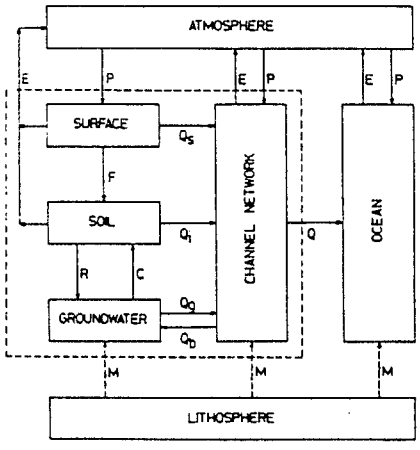


FIGURE 2.2 THE HYDROLOGICAL CYCLE IN SYSTEMS NOTATION
(Dooge 1973)

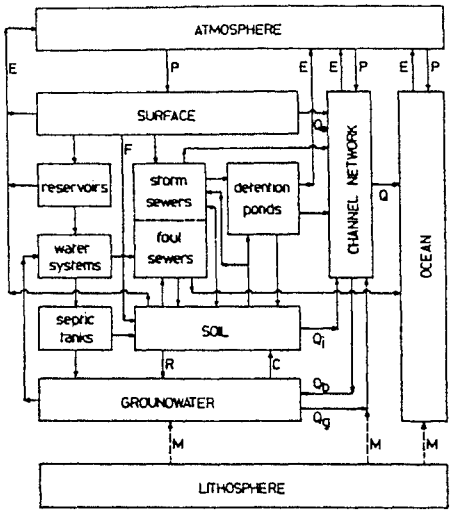


FIGURE 2.3 THE URBAN HYDROLOGICAL CYCLE
(Hall 1984)

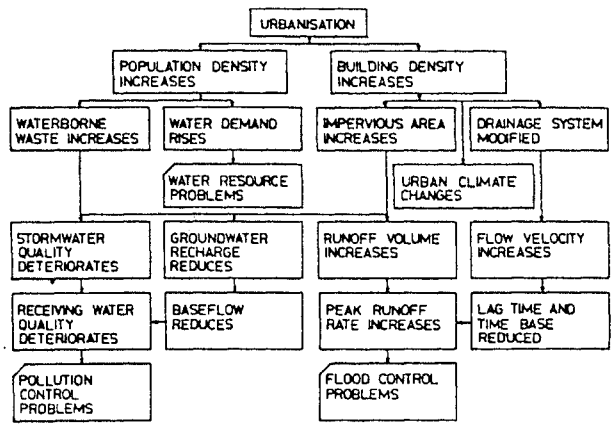


FIGURE 2.4 THE EFFECTS OF URBANISATION ON HYDROLOGICAL PROCESSES
(Hall 1984)

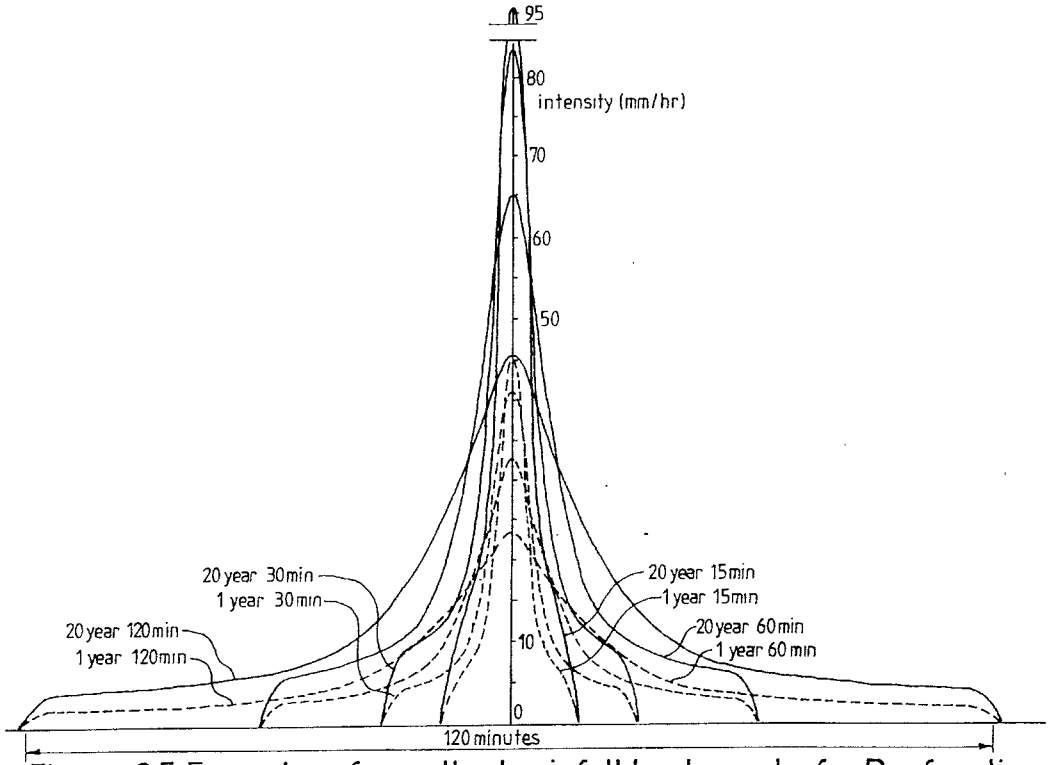
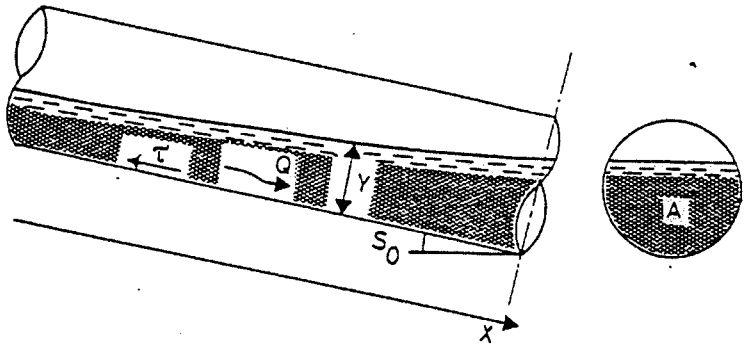


Figure 2.5 Examples of smoothed rainfall hyetographs for Dunfermline



$$\frac{\delta Q}{\delta x} + \frac{\delta A}{\delta t} = 0 \qquad (A)$$

$$\frac{1}{gA}\left[\frac{\delta Q}{\delta t} + \alpha' \frac{\delta}{\delta x}\left(\frac{Q^2}{A}\right)\right] + \frac{\delta y}{\delta x} = S_0 - \frac{\tau'}{\gamma g R} \qquad (B)$$

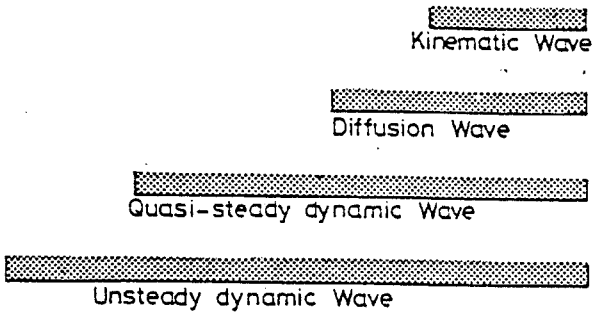


FIGURE 2.6 THE EQUATIONS OF CONTINUITY (A) AND OF MOTION(B) FOR UNSTEADY GRADUALLY VARIED OPEN CHANNEL FLOW (Modified from Jacobsen 1983)

CHAPTER 3
SEWER FLOW ROUTING METHODS

3.1 INTRODUCTION

Mathematical modelling of flow in sewers is rapidly becoming an accepted engineering tool, whose evolution can be compared to that of reduced physical scale modelling. Scale models came into use as design and verification tools when the complexity and scope of large structures began to present problems which could not be solved using traditional hydraulic methods, but could be accurately and productively modelled at a reduced scale (Howarth & Saul 1984, Saul et al 1984). The use of scale models and interpretation of their results provided important feedback into the development of theory such as similarity, statistics, wave motion and sediment movement as well as experimental science including measuring equipment and laboratory technique.

The theoretical foundations of physical scale models were laid down in the 1930s and 1940s. The general application of these models was mainly to solve open channel engineering problems. The early role of these physical models was to provide reliable quantitative results on which design decisions could be based (Cunge et al 1980). However, as engineering projects became larger, economic considerations were more and more often integrated into the overall planning of projects and hence scale models reached a natural limit to the scope of their application. Scale distortion together with engineering experience can sometimes extend the scope of scale models, however, there comes a point at which new techniques must be used to obtain representative results which are reliable and, most of all, economic. Mathematical modelling is one of these techniques.

Mathematical modelling of flow in sewers involves the simulation of flow conditions based on the formulation and solution of mathematical relationships expressing known hydraulic principles. The technique finds its origin in the 19th century work of de Saint Venant (1871) and Boussinesq (1877), who formulated the unsteady

flow equations, and in the work of Massau (1889), who published some early attempts to solve those equations. Despite the important theoretical concepts which had been established in the beginning of the 19th century, applications of the engineering principles were not widely available until the development of electronic computers. One of the first large scale applications was between 1952 and 1953 when a mathematical model was constructed for portions of the Ohio and Mississippi rivers (Isaacson et al 1954). Nowadays computer simulation models have become the most important analysis tool when dealing with complicated hydraulic problems.

3.2 THE UNSTEADY FLOW EQUATIONS

The fundamental notions and hypotheses used in the mathematical modelling of rivers are formalised in the equations of unsteady open channel flow. Two possible flow phenomena exist for river channels, these being channel flow and flood plain flow. Flood plain flow is significantly more complex and difficult to describe completely than channel flow. Most often the role of flood plains in flood propagation is to provide storage volume accompanied by a slow exchange of water from one part of the plain to another.

The flow of fluids in open channels is termed unsteady flow if conditions vary with time. Water flow in natural channels, such as rivers and reservoirs is nearly always unsteady, with only short periods of flow which may be considered steady. These flows may be placed into the two very broad categories of rapidly varied unsteady flows, such as waves following the collapse of a dam, and gradually varied unsteady flows, such as the propagation of flood waves. In many engineering applications, gradually varied unsteady flows may be treated as steady for the purpose of analysis, but conditions are properly described by two partial differential equations.

The first presentation of the partial differential equations of unsteady flow is attributed to Barre de Saint-Venant in 1871, following the contributions of his predecessors, notably Partiot,

Russell, Bazin, and Boussinesq. These equations are known universally as the Saint-Venant equations. More recently, increased sophistication due to the introduction of extra mathematical terms has not changed the basic mathematical representation of unsteady flow provided by the Saint-Venant equations. A number of methods exist for the derivation of the basic unsteady flow equations from shallow water theory, alternatively using concepts of energy slopes, or momentum change in a control volume. Some simpler derivations are given in the standard open channel flow texts (Chow 1959, Henderson 1966).

3.2.1 THE SAINT-VENANT EQUATIONS

The de Saint-Venant equations are generally shown in the following form (de Saint-Venant 1871) :

(i) The Continuity Equation

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \qquad (3.1)$$

where q is the lateral inflow per unit length of channel.

This equation is derived from the principle that the net rate of mass flow into a control volume equals the rate of change of mass storage in that volume.

(ii) The Momentum Equation

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\frac{\partial y}{\partial x} = gA\,(So - Sf) \qquad (3.2)$$

This equation is derived from the principle of the conservation of momentum applied to fluid flow (Newton's second law), which states that the net rate of change of momentum within a control volume equals the sum of the forces acting on that volume. The only three forces considered in this derivation are those due to gravity, frictional resistance and fluid pressure.

The Saint-Venant equations are described mathematically as two hyperbolic partial differential equations with two independent variables x (space) and t (time), and basically two dependent variables Q and y. The Saint-Venant equations (Eqns. 3.1 & 3.2) for unsteady flow are based upon the following assumptions :

(i) Vertical pressure distribution is hydrostatic.
(ii) Velocity variations in a channel cross-section are considered insignificant.
(iii) Wave movement is basically two-dimensional.
(iv) Average slope of the channel bottom is small.
(v) Unsteady flow friction losses are represented by empirical equations relating to steady flow.
(vi) Homogeneous flow i.e. constant fluid density ρ.

3.2.2 SIMPLIFIED METHODS OF SOLUTION

Analytical integration is only possible for a small number of simplified problems due to the complexity and non-linearity of the basic Equations 3.1 and 3.2. Numerical techniques must be employed to solve most practical problems of flood routing, and the large amount of repetitious arithmetic precludes the possibility of hand calculation.

Simplified methods are adequate and in many cases preferable when data are limited in quantity or quality, and high accuracy is not required. There are four broad categories of simplification :

(i) Use of the continuity equation alone,
(ii) Use of the continuity equation with a simplified version of the momentum equation,
(iii) Use of the momentum equation alone,
(iv) Use of processes not directly related to the unsteady flow equations.

Methods in Category (iv) above include techniques which rely on historical statistical information for the analysis of flow conditions. These methods utilise data such as rainfall and discharge from past flood events to predict future conditions, and

do not require the analysis of the physical processes involved in open channel flow. Statistical methods for analysing rainfall and flow data are well documented in hydrology textbooks, for example Linsley et al (1958), and in Volume I of Flood Studies Report (NERC 1975).

Methods based on (i) above rely on statistical records of past events to define the storage characteristics, and often use the storage differential form of the continuity equation. This states that the difference between inflow to, and outflow from, a reach is equal to the change in water storage in that reach :

$$\frac{dS}{dt} = I - O \qquad (3.3)$$

Methods based on the storage equations are numerous and conceptually simple, and do not directly include open channel flow theory. These methods are often refered to as hydrological methods.

A wave is defined as a recognisable signal that is transferred from one part of a medium to another with a recognisable velocity of propagation or celerity, it is hence suggested (Fread 1971) that wave theory is applicable to open channel in which flow is propagated downstream in the form of wave with recognisable speed.

Hydrological methods in general utilise records of past events to estimate the celerity of the flood wave and the storage characteristics within the reach. When inflow to the reach is greater than the outflow, for example, the water storage must be increasing. The concept of channel storage can be illustrated by an examination of the expanded form of the continuity equation :

$$A\frac{\partial v}{\partial x} + v\frac{\partial A}{\partial x} + B\frac{\partial y}{\partial t} = 0 \qquad (3.4)$$

A conceptual diagram for this equation is included as Figure 3.1. The first term represents prism storage in the reach, the second term wedge storage, and the third term is related to the rate of increase in depth. Storage characteristics of a reach depend on

the geometry and roughness of the channel and flood plain. Consequently estimation of these characteristics must emanate directly from flood records without knowledge of the hydraulic behaviour being required.

Methods in (ii) rely on the simplification of the momentum equation, which can be performed by any combinations of the techniques listed below :

(i) Ignoring the local acceleration $\partial Q/\partial t$;
(ii) Ignoring the convection of momentum $\partial/\partial x(Q^2/A)$;
(iii) Ignoring or linearising the resistance term.

Linearisation of the resistance term is carried out by replacing the quadratic term in Equation 3.2 between velocity and frictional resistance with a linear relationship.

Simplification of the momentum equation (Eqn. 3.2) by ignoring all terms except channel bottom slope So, and frictional resistance Sf, provides mathematical models based on the principles of kinematic flood routing. These methods can be shown also to be based on the convective-diffusion equation :

$$\frac{\partial y}{\partial t} + \omega \frac{\partial y}{\partial x} = \mu \frac{\partial^2 y}{\partial x^2} \qquad (3.5)$$

where ω and μ are constant parameters (Price 1973[1]).

Most methods based on simplification of the momentum equation are also based on equation 3.4. Fuller descriptions are included in the following Section 3.3.

Flood routing methods based on the convective-diffusion equation 3.5 are generally considered to provide greater accuracy than hydrological methods. A simplified flood routing model was introduced (Lighthill & Whitham 1955) based on a modification of the momentum equation. By neglecting all terms except the friction and channel bottom slope, the equations describe the movement of

kinematic waves as opposed to the dynamic waves which are described by the full equations. A kinematic wave is a wave of constant amplitude travelling with the same celerity as a monoclinal wave :

$$c = v + Ae \frac{dv}{dAe} \qquad (3.6)$$

Kinematic waves are based primarily on continuity considerations and terms relating to dynamic wave movement are ignored. Consequently they are able to travel only in a downstream direction and therefore contain only one velocity component.

Although convective-diffusion methods are generally more accurate than hydrological methods, they are also more complicated to apply in rivers and require more accurate flood data. Henderson (1966) has demonstrated the effect of the terms neglected from the momentum equation (Eqn. 3.2) on flood wave modification.

The above are some simplified methods to the full differential equations (Eqns. 3.1 & 3.2). If a greater accuracy than that provided by these simpler methods is required, a method based on the full unsteady flow equations must be considered.

3.3 NUMERICAL METHODS

Numerical integration of the full unsteady flow equations consists of replacing the differential equations with corresponding finite difference expressions, and solving these resulting equations. Development of a complete numerical model of an unsteady flow system requires the inclusion of channel geometry, roughness and the boundary conditions, which are normally stage hydrographs, discharge hydrographs or rating curve equations. The model is then calibrated by solving the equations with recorded boundary conditions and adjusting a channel parameter such as the roughness until adequate correlation is obtained between recorded and computed conditions.

The first numerical models were devised by Isaacson et al (1956, 1958), employing the explicit finite difference formulation (Stoker 1957). Solution techniques for the partial differential equations were adapted from the field of gas dynamics in which the basic equations are analogous to those of unsteady liquid flow. Following the work of Isaacson et al, a number of models have been developed for the study of both overland flow and varied open channel flow phenomena. Methods may be classified broadly as either characteristics methods based on the characteristic form of the equations, or direct finite difference methods based on the unsteady flow equations as originally derived (Eqns. 3.1 & 3.2). The finite difference representations used in the characteristic and direct methods may in turn be classified as either explicit or implicit, of which there are a wide variety of different basic formulations. Characteristics methods may also have a characteristic network or a fixed mesh, which according to Amein and Fang (1969), produces a total of six categories of numerical scheme :

(i) Explicit characteristics with characteristic network,
(ii) Implicit characteristics with characteristic network,
(iii) Explicit characteristics using fixed mesh,
(iv) Implicit characteristics using fixed mesh,
(v) Direct explicit,
(vi) Direct implicit.

A number of numerical schemes have been tried and rejected on the grounds of inaccuracy, instability or sometimes inpracticality (Cunge et al 1980). There is, however, still not any one method which is considered to be universally recognised as the best for every class of problem.

3.3.1 METHOD OF CHARACTERISTICS

The Method of Characteristics is fundamental to mathematicians for the study of many different problems. The method is based on the replacement of the two partial differential equations of unsteady flow (Eqns. 3.1 & 3.2) with four ordinary differential equations. Derivation of the application to open channel flow has been given

by many previous researchers (Henderson 1966, Abbott 1966 & 1975, Stoker 1957). A derivation is provided here for the simple case of a prismatic channel without lateral inflow or flood plain flow. Considering the two unsteady flow equations for a wide channel with depth y and velocity v as the dependent variables :

$$\text{Continuity :} \qquad v \frac{\partial y}{\partial x} + y \frac{\partial v}{\partial x} + \frac{\partial y}{\partial t} = 0 \qquad (3.7)$$

$$\text{Momentum :} \qquad \frac{1}{g} \frac{\partial v}{\partial t} + \frac{v}{g} \frac{\partial v}{\partial x} + \frac{\partial y}{\partial x} = So - Sf \qquad (3.8)$$

where the friction slope Sf may be defined by a suitable uniform flow expression.

The celerity c of a small wave is substituted into the equation as a parameter representing the water depth y :

$$c^2 = gy \qquad (3.9)$$

also noting that the differential :

$$d(c^2) = d(gy) = 2c\ dc \qquad (3.10)$$

The result of these substitutions and manipulation of the equations is a set of four ordinary differential equations :

$$\frac{dx}{dt} = v \pm c \qquad (3.11)$$

$$\left\{ (v \pm c) \frac{\partial}{\partial x} + \frac{\partial}{\partial t} \right\} (v \pm 2c) = g(So - Sf) \qquad (3.12)$$

The two equations represented by positive and negative c in Equation 3.11 are the equations of the forward (α) and backward (β) characteristic curves defined on an x, t plane in Figure 3.2. Equations 3.12 define respectively how parameters, in this case depth y and velocity v, vary along the characteristic curves. Figure 3.2 also shows that for initial known points on the plane, the characteristics issued from these points and the solutions to

Equations 3.11 and 3.12 are defined by the intersections of the characteristic curves. Boundary conditions are specified in a similar way to other methods such as discharge hydrographs, stage hydrograph and rating curve, and conditions at internal points are found from the solution of the boundary conditions and characteristic equations at the intersection of the characteristic curves. Figure 3.2 is also a characteristic representation of an unsteady subcritical flow.

Characteristic methods have been utilised for the solution of many problems in the field of hydraulics. A number of examples have been given by Fox (1977) for solving problems of unsteady flow in pipe networks, including water hammer and surge tank analysis, as well as free surface flow in sewers.

Numerical schemes for characteristics method have been limited by the fact that results are obtained on the distance/time (x,t) grid at uneven intervals. Solution of the equations is obtained at the intersection of the characteristics curves, and not on a fixed grid as with the direct methods. Results therefore require to be interpolated between points defined on the characteristic grid to provide the values of the dependent variables required (e.g. depth and velocity) at a desired location in time and space. The need for interpolation has been cited as a criticism of characteristic methods. However this is counteracted by the fact that the characteristic grid becomes more closely spaced in regions of rapid change, and of course tightly spaced grid points assist in accurate computation.

It has been shown above that the method of characteristics requires a closely spaced grid. This results in a greater amount of interpolation than other methods. Fixed mesh characteristic methods, for example, have been developed which reduce the amount of interpolation required. It can be seen from Figure 3.3 that depth and discharge at point M are obtained from the conditions at points B and D, which are found from simple interpolation of the known conditions at points A, C and E. An example of the use of this type of fixed mesh is described by Miller and Cunge (1975).

3.3.2 EXPLICIT METHODS

Explicit methods are so called because they are solved explicitly for flow parameters at one point at a time. Figure 3.4 shows that conditions at point (i, j+1) may be found from the known conditions on row j independently of any other conditions at the new time level on row j+1. Hence numerical schemes and algorithms are relatively simple, and consequently explicit schemes were used extensively in early numerical models until their inherent limitations generated the requirement for alternative methods.

A number of explicit methods have been developed, differing primarily in the finite difference representation of the differential equations. Four most popular methods are listed in the following :

(i) Unstable,
(ii) Leap-frog,
(iii) Diffusive,
(iv) Lax-Wendroff.

There are many more methods available but it is beyond the scope of this research to detail all the difference equations generated by each method. However a brief account of each of the above is included, with particular reference to the accuracy and stability of each scheme.

Stability analysis generally comprises expressing the solution of the linearised finite difference equations as a Fourier Series (Liggett & Cunge 1975). A necessary but insufficient condition for stability known as the 'Courant-Friederichs-Lewy' criterion, or more often the 'Courant' condition (Richtmyer & Morton 1967) restricts the size of the time step which may be chosen for all explicit methods.

The Courant condition can be stated mathematically :

$$\frac{\Delta t}{\Delta x} \leq \frac{1}{\frac{Q}{Ae} + \left(g \frac{Ae}{B}\right)^{1/2}_{max}} \qquad (3.13)$$

where Δx is the distance step chosen for the model and Δt is the time step. Referring to Figure 3.3, the condition means that point M must be within the characteristic curves emanating from points A and E, which ensures conditions at point M are dependent on conditions at points A and E. Equation 3.13 indicates that the time step used in the computation is governed by the celerity of dynamic waves in a channel.

The unstable method includes the finite difference scheme which at first sight is the most obvious. Again with reference to Figure 3.4, the finite difference representation of the partial differentials of a dependent variable such as discharge Q, may be written as :

$$\frac{\partial Q}{\partial x} = \frac{Q^{j}_{i+1} - Q^{j}_{i-1}}{2\,\Delta x} \quad ; \quad \frac{\partial Q}{\partial t} = \frac{Q^{j+1}_{i} - Q^{j}_{i}}{\Delta t} \qquad (3.14)$$

Equation 3.14 employs a centred difference scheme for the space derivative and a forward difference scheme for the time derivative. The inherent instability of the method has been criticised by many researchers (Liggett & Woolhiser 1967) simply because the method has to satisfy the Courant condition.

Models based on the explicit method normally employ the Leap-frog, Diffusive or Lax-Wendroff schemes which are inherently more stable. Again with reference to Figure 3.4, the time derivative for the diffusive scheme is written :

$$\frac{\partial Q}{\partial t} = Q^{j+1}_{i} - \left\{ \Theta\, Q^{j}_{i} + \frac{1-\Theta}{2} \left(Q^{j}_{i-1} + Q^{j}_{i+1} \right) \right\} \qquad (3.15)$$

with the space derivative unchanged. For $\Theta = 0$, a fully diffusive scheme is obtained whilst for $\Theta = 1.0$ gives the unstable method.

Leap-frog schemes employ centred differences in both space and time, and are the most commonly used explicit scheme. Referring to Figure 3.4 :

$$\frac{\partial Q}{\partial x} = \frac{Q_{i+1}^{j} - Q_{i-1}^{j}}{2\Delta x} \quad ; \quad \frac{\partial Q}{\partial t} = \frac{Q_{i}^{j+1} - Q_{i}^{j-1}}{2\Delta t} \qquad (3.16)$$

Both the Diffusive and Leap-frog schemes must comply with the Courant condition for stability. To obtain the optimum accuracy the time step Δtc is defined as follows :

$$\frac{\Delta tc}{\Delta x} = \frac{1}{\frac{Q}{Ae} \pm \left(g \frac{Ae}{B}\right)^{1/2}} \qquad (3.17)$$

It is impossible for the time step to satisfy Equation 3.17 for all points in a reach for the duration of an event, but this equation gives guidance for selection of time step intervals.

The Diffusive and Leap-frog schemes may exhibit saw-tooth fluctuations in space in the output. This phenomenon, which must be considered to be different from numerical instability, may prove detrimental to a model but can normally be eradicated.

The Lax-Wendroff scheme provides second order accuracy with the finite difference expression given by Richtmyer and Morton (1967). Similar fluctuations may occur with this method, particularly for analysis of rapidly varying flow conditions.

In addition to their application to various open channel flow problems, explicit methods are also used in the study of tidal rivers, estuaries and seas where rapidly varying conditions dictate a small time step and the Courant condition is not limiting.

Explicit schemes have also been used extensively for two- and three-dimensional models because of the relative simplicity of the finite difference representation.

3.3.3 IMPLICIT METHODS

Implicit schemes were first developed in the early 1960s because of the prohibitive restriction on the time step size when using explicit schemes. A number of different implicit schemes have subsequently been developed but they all adhere to one fundamental principle. The following refers back to Figure 3.4. If conditions at all nodes on row j are known, then conditions after one time step Δt on row j+1 are found from the simultaneous solution of all the finite difference equations for the two rows, and the boundary conditions. The dependent variables, stage (y) and discharge (Q), are contained implicitly in the non-linear difference equations and hence solutions must involve an iterative technique of the equation.

Researchers using implicit methods have tended to develop their own methods for specific problems and therefore many implicit methods exist which differ in the finite difference discretisation or in the algorithm. However, three methods stand out as being the basis of most other methods and as being well used by large research organisations. They are :

(i) Preissman (SOGREAH) scheme,
(ii) Amein's four-point scheme,
(iii) Abbott's scheme.

One of the earliest methods was developed by Preissman of SOGREAH, France, and the finite difference expressions are presented by Cunge and Wegner (1964). Using the layout in Figure 3.4, the partial differentials of discharge for advancing from row j to row j+1 may be written :

$$\frac{\partial Q}{\partial t} = \frac{1}{\Delta t}\left\{ \frac{Q(A) + Q(B)}{2} - \frac{Q(C) + Q(D)}{2} \right\} \qquad (3.18)$$

$$\frac{\partial Q}{\partial x} = \frac{1}{\Delta x}\left\{ \Theta \langle Q(B)-Q(A)\rangle + (1-\Theta)\langle Q(D)-Q(C)\rangle \right\} \qquad (3.19)$$

where Θ is a finite difference weighting coefficient having a value between 0.0 and 1.0 which affects the numerical stability and accuracy of the scheme. As the four nodes A, B, C and D define the four corners of a box, this implicit method is sometimes referred to as the 'box scheme'.

Development of the method progresses by replacing $\partial Q/\partial t$, $\partial Q/\partial x$ and the corresponding expressions for stage y in the continuity and momentum equations (Eqns. 3.1 & 3.2), by their representative finite difference forms (Eqns. 3.18 & 3.19). Preissmann continued by linearising the resulting equations in the dependent variables Q and y at the new time level (row j+1).

Two simultaneous equations with four unkowns are generated for each 'box' which have the following form :

$$C1\ y(A) + C2\ Q(A) + C3\ y(B) + C4\ Q(B) = C5 \qquad (3.20)$$

$$C6\ y(A) + C7\ Q(A) + C8\ y(B) + C9\ Q(B) = C10 \qquad (3.21)$$

where C1 - C10 are dependent only on conditions at the old time level (row j) and hence do not contain the values of Q and y to be calculated.

If a similar pair of equations are defined for each 'box' between row j and row j+1 and boundary conditions are included for each end of the reach, a system of 2N equations with 2N unkowns is obtained, where N is the number of space nodes. These linear equations may then be solved for the dependent variables at the new time level, by a process such as Gaussian elimination.

Amein's method is similar to that of Preissmann and utilises the same four-point 'box' scheme for defining the finite difference expressions. Early versions of this method (Amein 1968, Amein & Fang 1969 & 1970) used a value of $\Theta = 0.5$ in Equation 3.19 which produces a centred difference scheme in both space and time. A

later version of the method employed a value of $\theta = 1.0$ (Amein & Chu 1975) which produces a forward difference scheme in time and greatly simplifies the equation.

The fundamental difference between Amein's and Preissmann's scheme is in the treatment of the finite difference equations. Amein does not linearise the basic finite difference expressions and hence the resulting equations are non-linear in the dependent variables Q and y. An iterative technique such as a generalised Newton-Raphson method must therefore be used to solve the 2N simultaneous equations with 2N unknowns.

The third implicit method utilises a six point finite difference discretisation. This method is usually attributed to Abbott (Abbott & Ionescu 1967) although it may also be referred to as the Vasiliev implicit scheme. Verwey (1970) describes the method as a double tri-diagonal implicit scheme solved with a double sweep algorithm. The finite difference representation, using Figure 3.4, of the partial differentials of a dependent variable Q may be written :

$$\frac{\partial Q}{\partial t} = \frac{Q_i^{j+1} - Q_i^j}{\Delta t} \qquad (3.22)$$

$$\frac{\partial Q}{\partial x} = \frac{1}{2\Delta x}\left\{\frac{1}{2}(Q_{i+1}^{j+1} + Q_{i+1}^{j}) - \frac{1}{2}(Q_{i-1}^{j+1} + Q_{i-1}^{j})\right\} \qquad (3.23)$$

The above partial differential expressions are based on a six-point grid leading to its description as a six-point method.

A further difference between Abbott's scheme and the four-point schemes mentioned is that the values of Q and y are calculated at alternate space along the reach, which defines two simultaneous staggered finite difference grids. The continuity equation is centred on the y points and the momentum equation is centred on the Q points.

A number of analyses have been carried out of the stability and accuracy of implicit methods. Unlike explicit methods, the choice of time step size in implicit method applications is unaffected by the Courant condition and hence in many cases unrealistically large time steps will not adversely affect the stability of the method. It has, however, been demonstrated by many researchers (Cunge & Wegner 1964, Abbott & Ionescu 1967, Bettess & Price 1976, Shepherd 1979) that stability in no way guarantees accuracy, and that exceptionally large time steps render the implicit models worthless.

3.3.4 COMPARISON OF NUMERICAL METHODS

A particular unsteady flow problem may be solved adequately by a number of different numerical techniques. Similarly a particular numerical technique may be adapted to provide stable and accurate solutions for a number of different unsteady flow problems.

Bettess and Price (1976) carried out a study of a number of different methods for numerically modelling flood waves down part-full pipes. Their aim was to discover which methods were most suitable, with particular reference to accuracy and speed of computation. Seven numerical schemes were compared for speed and accuracy on a number of different synthetic storms, these being :

(i) Four-point implicit method,
(ii) A linearised version of the four-point implicit method,
(iii) Lax-Wendroff method,
(iv) A method based on linear characteristics,
(v) A convective-diffusion method,
(vi) The Muskingum-Cunge method,
(vii) A non-linear kinematic wave method.

The following conclusions were obtained when the seven methods were

compared (Bettess & Price 1976) :

(i) The Muskingum-Cunge method was comparable in accuracy with the method based on the convective-diffusion, but computationally much faster ;

(ii) The non-linear kinematic wave theory was less accurate than the other two kinematic methods ;

(iii) The Lax-Wendroff method and the linear characteristic method were both considerably restricted because of stability requirements and computation was extremely slow ;

(iv) The linearised implicit method was computationally the fastest of the four schemes that approximated to the Saint-Venant equations and also provided an acceptable degree of accuracy.

The Muskingum-Cunge method was the one recommended to be used in the analysis of storm sewer systems following these results and for the simulation of flows in such systems where speed of computation is an important factor.

Simulation model DUCTS for the research study utilises the Muskingum-Cunge method in dealing with the free surface flow routing in pipes. The enhanced parallel pipe model DUPPERS also has this procedure retained in the computational procedure to route flows in the twin pipes system. However, the constant parameters computed in both the models are investigated based on the suggested acceptable ranges which have been detailed in the next section.

3.4 THE MUSKINGUM-CUNGE METHOD

Bettess and Price (1976) showed that the Muskingum-Cunge method is considered to be one of the simplest flood routing methods yet still performs satisfactorily. It was recommended for the simulation of flows in storm sewer systems. The method also has the advantage over many others in that it includes an allowance for dynamic storage.

The basic Muskingum method was originally developed by McCarthy (1938) for the US Army Corps of Engineers and derives its name from its use in the study of flood control schemes for the Muskingum River in Ohio in 1935 and has been used by river engineers in a fundamentally similar form ever since. The method was later improved by Cunge (1969) and used in flood routing both in rivers and in sewer systems (Price 1973[2], 1978) and referred to as the Muskingum-Cunge Method.

3.4.1 BASIC MUSKINGUM THEORY

The Muskingum method described by McCarthy in 1938 is a hydrological method based on the finite difference form of the storage equation :

$$\left\{ \frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} \right\} \Delta t = S_2 - S_1 \qquad (3.24)$$

where I, O and S are inflow, outflow and storage respectively for a reach, and subscripts 1 and 2 refer to conditions before and after a prescribed time period Δt.

The storage produced during the advance of a flood wave is wedge-shaped (Figure 3.1), as the inflow normally exceeds the outflow. The wedge can be related to the difference between the instantaneous values of inflow and outflow. The wedge storage here is represented by the term $K\xi(I-O)$ whilst the additional prism storage corresponding to steady flow is shown as the term KO. Summing the two together gives :

$$S = KO + K\xi\ (I - O) \qquad (3.25)$$

Equation 3.25 is normally shown in the form of :

$$S = K\ [\ \xi\ I + (1 - \xi)O\ ] \qquad (3.26)$$

where K and ξ are the two Muskingum parameters defining the channel or sewer characteristics. K is termed the storage parameter and is

related to the speed of travel of flood waves along the reach, and ξ is a weighting parameter relating the comparative effect of inflow and outflow on channel storage. Combination of Equation 3.24 and the finite difference expression of Equation 3.26 provides the basic Muskingum equation which is explicit in the unknown outflow O_2 :

$$O_2 = C_1 I_1 + C_2 I_2 + C_3 O_1 \qquad (3.27)$$

where :

$$C_1 = \frac{K\xi + 0.5\,\Delta t}{K(1-\xi) + 0.5\,\Delta t}$$

$$C_2 = \frac{-K\xi + 0.5\,\Delta t}{K(1-\xi) + 0.5\,\Delta t}$$

$$C_3 = \frac{K(1-\xi) - 0.5\,\Delta t}{K(1-\xi) + 0.5\,\Delta t}$$

In the original method, the coefficients K and ξ are calculated exclusively from recorded discharge hydrographs at the upstream and downstream boundaries for past flood events. Although this method was based on a unique stage-discharge relationship, the method was able to model the attenuation of a flood peak. Cunge (1969) showed how these conflicting results could be resolved.

3.4.2 IMPROVEMENTS BY CUNGE

Cunge (1969) radically improved the basic Muskingum method by permitting the calculation of the Muskingum parameters directly from the channel characteristics, effectively producing a method based on the convective-diffusion equation. Cunge criticised the Muskingum method for being based on an unique relationship between depth and discharge. As no attenuation would occur theoretically under this condition, attentuation and time lag predicted are dependent primarily on errors introduced by the finite difference representations of the analytical equations, rather than modelled physical conditions.

Firstly Cunge considered the kinematic wave equation in the form of :

$$\frac{\partial Q}{\partial t} + \omega \frac{\partial Q}{\partial x} = 0 \qquad (3.28)$$

where ω is the kinematic wavespeed and proposed a finite difference representation of this equation using a rectangular x-t grid (Figure 3.5).

By combining the original Muskingum equations (Eqns. 3.24 & 3.26) over a finite time step, Cunge then derived a suitable expression for the weighting parameter ξ by expanding the combined equation as a Taylor Series, and showing that this expression is also a finite difference representation of a general convective-diffusion equation :

$$\frac{\partial Q}{\partial t} + \omega \frac{\partial Q}{\partial x} = \mu \frac{\partial^2 Q}{\partial x^2} \qquad (3.29)$$

when μ is defined by :

$$\mu = (1/2 - \xi) \omega \Delta x \qquad (3.30)$$

Cunge then defined ξ in terms of channel top width and average slope. Subsequently Price (1973[1]) obtained an expression for the diffusion coefficient μ based on the work of Hayami (1951) which provides an algebraic expression for ξ related to physical conditions :

$$\xi = \frac{1}{2} - \frac{\alpha \bar{Q}p}{L \omega \Delta x} \qquad (3.31)$$

where α is an attenuation parameter corresponding to an average peak discharge Qp, with L representing the total length of channel. The attenuation parameter α is defined (Price 1973[1]) in terms of channel and flood plain characteristics, with ω and Qp obtained from records of past flood events.

Parameters K and ξ in the Muskingum-Cunge Method are therefore related directly to physical conditions, unlike the corresponding parameters in the Muskingum Method, but results are still dependent on the finite difference grid employed (Figure 3.5).

3.4.3 CHOICE OF MUSKINGUM-CUNGE PARAMETERS FOR PIPE FLOW ROUTING

Although the Muskingum-Cunge method was developed for the channel flow, it could also apply to pipe flow under free surface condition. The major advantage of the Muskingum-Cunge method is that few data are required for calibration and application. Using the Muskingum-Cunge method, K and ξ are defined from α which can be related to the pipe characteristics, and ω, the kinematic wavespeed. Other coefficients in the method are made up of the parameters Δt, Δx, α and values of these parameters may be selected in a number of ways producing many different arrangements.

In the original Muskingum Method the space increment Δx was made equal to the total length of the routing reach. The total reach length was later subdivided into 'n' sub-reaches of length Δx in order to obtain better accuracy. The number of sub-reaches n and the weighting parameter ξ were not independent of each other, but the optimum value of ξ decreased with increasing n. A maximum number of sub-reaches is reached when ξ is equal to 0 while the minimum number depends on channel and hydrograph characteristics.

More recent work on the choice of the space increment has been related to the accuracy of the Muskingum-Cunge Method for both wave speed and attenuation. Jones (1981) produced a graphical relationship of contours of constant error against $1/\omega r$ (Figure 3.6) enabling a convenient form of a restriction on these parameters to be selected. He stated that in order to achieve an accuracy of within 5% for both wavespeed and attenuation the

following two conditions were necessary :

$$0.115 < \xi < 0.5$$

and $$0 < 1/\omega r < 1.6 \qquad (3.32)$$

From Equation 3.30, this condition requires that :

$$2.6\ \mu/\omega \leq \Delta x \leq 1.6\ \omega\ \Delta t \qquad (3.33)$$

Price (1981) later revised these restrictions and concluded that to achieve an accuracy of within 5%, it was adequate to take :

$$1/\omega r \leq 1.6$$

which gives $$\Delta x \leq 1.6\ \omega\ \Delta t \qquad (3.34)$$

Cunge (1969) showed that for all values of $\Delta x/\Delta t$, a value of the weighting coefficient $\xi = 0.5$ resulted in pure translation of the wave. Similarly a value of $\xi = 0$ corresponded to reservoir type storage where inflow caused an instantaneous response, the principal effect being attenuation of the inflow peak. By choosing an appropriate value for ξ between the two extremes, the phenomena of translation and attenuation can be treated simultaneously.

A representative value for the wavespeed can be found from the following equation (Price & Kidd 1978) :

$$\bar{\omega} = \frac{1}{Qfb} \int_0^{Qfb} \frac{dQ}{dA}\, dQ = \frac{1}{Qfb} \int_0^{d} \frac{1}{B} \left(\frac{dQ}{dy} \right)^2 dy \qquad (3.35)$$

With Q being defined by the normal depth relationship of the Colebrook-White equation :

$$Q = A\ (32\ g\ R\ So)^{1/2}\ \log_{10} \left(\frac{14.8\ R}{ks} \right) \qquad (3.36)$$

A water depth of 0.6 pipe diameter was chosen for the model DUCTS and DUPPERS for the evalution of Q, B and ω as, it was suggested (Jones 1981), this gave an approximate value for ξ. Before the models were used for simulation, the sewer length (Δx) and the chosen time step (Δt) used in the models were compared with those suggested by Jones and the regions in Figure 3.6. The computed Muskingum-Cunge constants are also printed out in the checking output datafile for the inspection of negative values.

3.5 REVIEW OF FLOW ROUTING METHODS

The various flow routing schemes and available solution techniques have been reviewed to support the required improvements for the flow routing procedure in the parallel pipe model DUPPERS in the free-surface flow condition. This understanding of the flow routing methods has also enabled the level computation for free-surface flow to be developed easily in the enhanced DUPPERS model.

The conceptual model DUCTS was developed based upon the available published documents such as IOH reports (IOH 1974-1979) but was lacking calibration using real catchment data. The performance of a flow routing procedure for free-surface conditions, therefore, could not be justified. However, the procedure for solving the surcharged flows in DUCTS was better developed than the free-surface flows in DUCTS and the surcharged sub-system formation method had been identified and checked using a separate computer program (Ashley & Jefferies 1986).

The major enhancements for the parallel pipe model DUPPERS focussed on the under ground flow routing procedure and hence the performance of the routing methods used in the model required to be identified. Before any improvement tasks commenced, some tests on the routing procedure were performed so that the performance of the methods for both free-surface and pressurised flows could be

established (see Chapter 7). The computation of the routing constants such as those for the Muskingum-Cunge equations were checked in order to determine whether the time and space intervals were correctly chosen as suggested in Section 3.4.3.

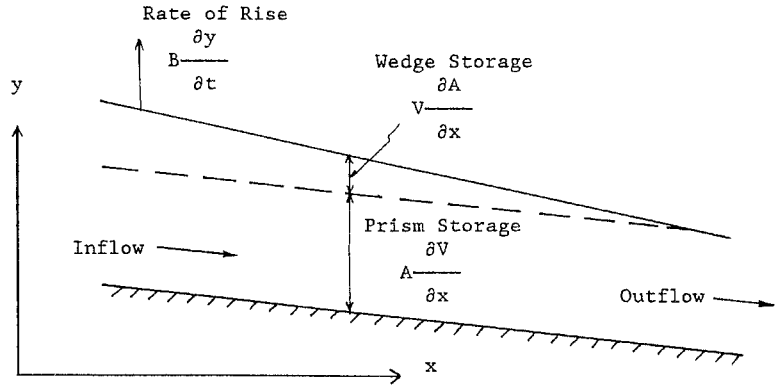


FIGURE 3.1 REPRESENTATION OF THE CONTINUITY EQUATION

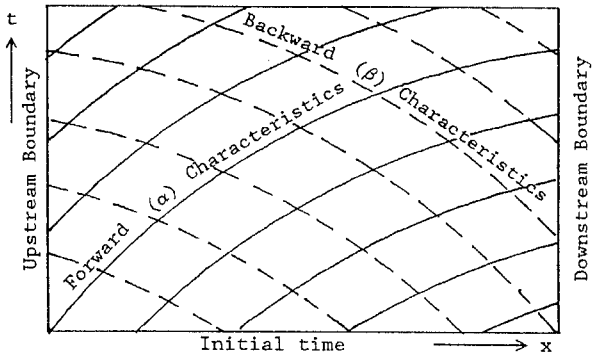


FIGURE 3.2 CHARACTERISTICS SPACE / TIME GRID

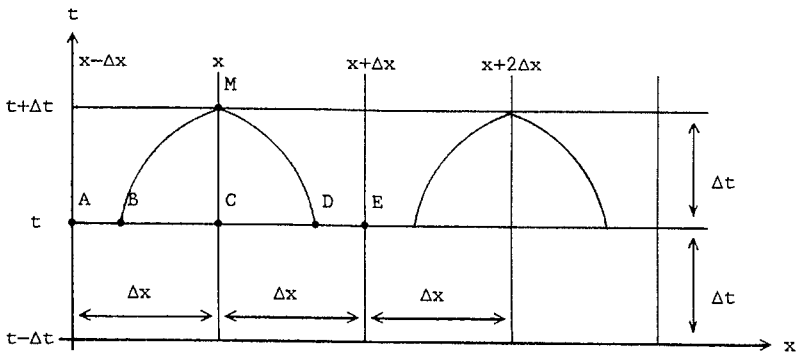


FIGURE 3.3 RECTANGULAR GRID FOR THE CHARACTERISTIC METHOD

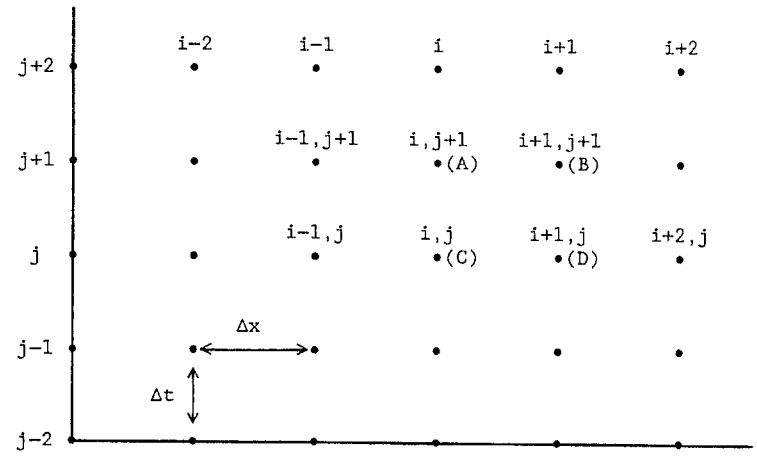


FIGURE 3.4 FIXED RECTANGULAR SPACE / TIME GRID

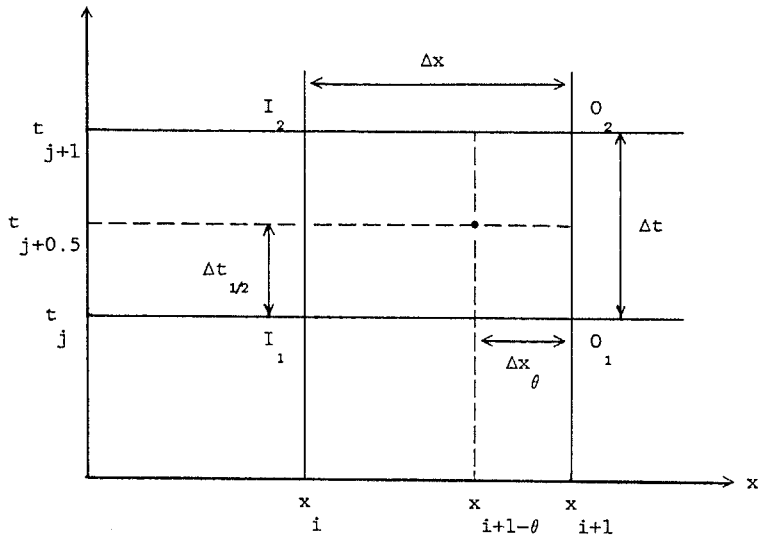


FIGURE 3.5 FINITE DIFFERENCE GRID FOR KINEMATIC MODELS

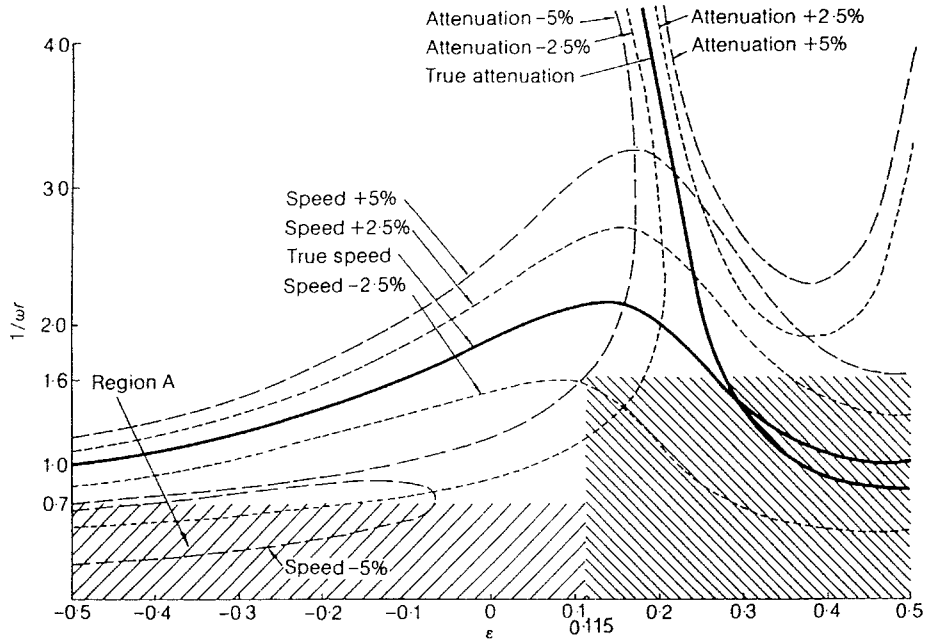


FIGURE 3.6 ERROR CURVES SHOWING RECOMMENDED RESTRICTIONS ON ξ AND $1/\omega r$ (Jones 1981)

CHAPTER 4
CONCEPTUAL SIMULATION MODELS OF URBAN CATCHMENTS

4.1 INTRODUCTION

When a catchment area is urbanised and the amount of impervious cover in the form of roofs, roads and pavements increases, the need inevitably arises for the natural drainage network to be supplemented or even replaced completely by man-made systems of pipes and paved gutters. These systems of pipes or sewers generally assume a dendritic form in plan, similar to that of a network of natural channels. However, the hydrological design problems associated with sewerage, i.e. systems of sewers, differ from those concerned with channel works in that no measurements of surface water runoff are possible prior to construction. Design flood estimates for sewers must therefore be inferred from rainfall statistics using deterministic methods. As a corollary to this design approach, the performance of a sewerage system once constructed is rarely recorded unless problems are encountered with its behaviour under conditions approaching those of the design storm. This lack of incentive, coupled with the difficulties of gauging flows in sewers, has resulted in a dearth of flow records from sewered catchment areas, which has perhaps provided the biggest obstacle to the development of stormwater drainage design methods.

Sewerage systems, may be broadly classified into two types, namely:

(i) combined systems, in which both the stormwater drainage and the domestic waste or sewage are conveyed in the same pipe network; and
(ii) separate systems, in which the foul drainage is conveyed to the nearest treatment plant and the stormwater drainage is carried in its own system of sewers to the nearest watercourse.

In practice, partially combined systems are to be found, which carry the domestic sewage from only a proportion of the area. However, the

stormwater discharge can be many orders of magnitude larger than the so-called dry-weather flow, and therefore provides the more dominant design consideration.

The flood estimation methods which have been applied to the design of stormwater drainage systems may be considered to fall into two broad categories: those which produce only an estimate of the peak flow rate, and the more comprehensive approaches that also provide the shape of the runoff hydrograph. With the wider availability of digital computers, the design hydrograph methods have increased in their scope and complexity. These later developments, which are distinguished primarily by the separate modelling of the above-ground and the below-ground phases of runoff, have been discussed in Chapter 2.

In recent years, practising engineers in the UK have expressed concern about the tools available for the design of storm sewer systems (Ashley et al 1989). This concern was most clearly stated in 1973 (CIRIA 1974) and has resulted in an investigation at the Hydraulics Research Station into design methods. The investigation includes an examination of methods available and other related literature (Colyer & Pethick 1976), a comparison of the performance of some existing methods, and the development of new methods (Colyer 1977). Other worldwide methods were also investigated with regard to their accuracies and application in storm water drainage and pollution simulation (Zaghloul 1977).

The problems of urban drainage design can range from the analysis of existing sewer networks to the design of entirely new systems, and the area served may vary in size from a small housing estate to a large conurbation. In order to cover the wide range of possibilities which occur, a design procedure incorporating a hierarchy of methods is required, similar to that for the estimation of floods on natural catchment areas. The application of this concept to stormwater drainage design is conveniently illustrated by the Wallingford Procedure, the details of which are outlined in this chapter. Due to the limit of time, only the Wallingford Procedure and DUCTS (Dundee College of Technology Sewer Simulation) models have been employed for the study of the Lyneburn sewerage system. Nevertheless, there are

many other well-presented models and a number are outlined in the following section. Most are specially tailored computer methods developed for particular study areas.

As well as those described below, there are two other well known simulators which could have been considered, but have not been described due to a lack of available information. The first of these is CAREDAS (Brandstetter 1975, Chevereau et al 1978) which was developed by Sogreah in 1973-74, and is in use in France today (Cunge & Mazaudou 1984) to deal with large and complex storm sewer systems. The other is a newer model called MOUSE from Denmark (DHI 1987) which has a better representation when dealing with the above ground phase and is also capable of simulating fully looped networks by providing dynamic solution (Chapman 1990). On-screen graphical displays are another versatile means used to interpret the solutions provided by MOUSE.

4.2 CHICAGO HYDROGRAPH METHOD

One of the first sewer design methods in which the above-ground and below-ground phases of the rainfall-runoff process were treated separately was the Chicago Hydrograph Method, a description of which was presented by Tholin and Keifer (1960). As its title implies, this method was devised specifically for use in the Chicago area where main sewers were generally laid out in parallel at intervals of about 0.8km, and the lateral sewers served sub-areas of approximately 25ha. The method was based upon a 3h, once-in-5-year design storm derived from local rainfall records. The transformation of this storm into a runoff hydrograph began with the abstraction of infiltration losses to give a hydrograph of overland flow and depression storage supply. After deducting an allowance for depression storage, the hydrograph was routed through channel storage to give the variation of flow with time at the nearest road gulley. The below-ground component of the rainfall-runoff model then began with the routing of the gulley hydrographs through the lateral sewer serving each sub-area. If the discharge hydrograph from a group of sub-areas was required, flows from the lateral sewers could then be routed down a main sewer.

The layout of both lateral and main sewers and the uniformity of the sub-area geometry and land use served to reduce the amount of calculation involved in the Chicago Hydrograph Method. Even so, the approach was too laborious for hand calculations, and a digital computer was employed to produce a series of design charts which Tholin and Keifer claimed had made the method as straightforward in application as the Rational Method. However, with the wider availability of computer facilities, this intermediate step of preparing design charts has been superseded by more direct interactive computer use.

The calculation procedure for the Chicago Method and the data and computational requirements may be found in the literature review written by Colyer and Pethick (1976).

4.3 UNIVERSITY OF CINCINNATI URBAN RUNOFF MODEL (UCURM)

The University of Cincinnati Urban Runoff Model (UCURM) described by Papadakis and Preul (1972) is similar in form to SWMM which will be covered in a later section. However, UCURM depends upon the division of the catchment into sub-areas that are either completely pervious or completely impervious. On each of the latter, allowances are made for infiltration and surface retention together with overland and channel flow before routing the gulley hydrographs through the pipe system. In its original form, UCURM was criticised by Heeps and Mein (1973, 1974) for its over-simplified approach to the treatment of infiltration and depression storage on pervious sub-areas, and the inefficiency of a solution procedure for the routing of overland flow. Replacement of the latter by an iterative technique achieved a remarkable reduction in computer central processor time to 4% of the original figure.

Although UCURM cannot be considered to have progressed very far beyond the initial development stage at the time of the study by Heeps and Mein (1973, 1974), the results obtained by the latter authors serve to illustrate the care which must be exercised in formulating rainfall-runoff models and programming their solution. In view of the aims of the project carried out by Heeps and Mein,

more attention was paid to the below-ground phase of runoff, and the relatively simple geometry of the pipe network makes the use of physically based models a practical possibility. Comparative studies by Yevjevich and Barnes (1970), Cunge (1974), Yen and Sevuk (1975) and Zaghloul (1977) amongst others have shown that simplified methods of routing are adequate for small pipe networks, but for larger systems, where backwater effects may be important, solution procedures based upon the full unsteady flow equations are necessary.

4.4 TRANSPORT AND ROAD RESEARCH LABORATORY HYDROGRAPH METHOD (TRRL)

The TRRL method (Watkins 1962, 1970) is a conceptually simple model compared with other mathematical models, and has been used extensively as a design tool in Great Britain. Two striking features of the model are that the areas which contribute storm runoff are taken to be only those impervious areas directly connected to the pipe system and have a runoff coefficient of 100%. The former assumption has attracted most criticism of the model (Jones 1970, Linsley 1970, Snyder 1970).

Overland flow on these contributing areas is simulated by combining the rainfall hyetograph and a time versus contributing area diagram (time-area routing) to give an inflow hydrograph to the pipe under consideration. The RRL method computer program supplied by the Transport and Road Research Laboratory [as distinct from the description given by Papadakis and Preul (1973)] assumes a linear time versus contributing area diagram for each inlet. The whole area contributes after the time of entry plus the time of travel at full-bore flow as calculated by the Colebrook-White formula. The time of entry at an inlet is the time required for all the directly connected impervious area to contribute to runoff. It is assumed constant for any inlet and must be estimated externally and included as part of the input data.

A unique feature at the time of development of the TRRL method was its ability to design pipe diameters to avoid surcharging. This is

carried out by successively increasing the pipe diameter and repeating the calculations until the peak of the outflow hydrograph (after storage routing) does not exceed the capacity of the pipe.

To clarify the differences between the original version (Papadakis & Preul 1973, Terstriep & Stall 1969, Watkins 1962) and the modified version (Heeps & Mein 1973, Stall & Terstriep 1974, Watkins 1970), it is worth restating the following points about the modified version:

(i) The TRRL method computes flow, pipe by pipe, from the most remote pipe in the drainage basin to the outfall.
(ii) The time-area diagram for each inlet is assumed to be linear.
(iii) No allowance is made for surface storage.
(iv) Storage routing to account for pipe detention storage is performed at each pipe in the system.

4.5 ILLINOIS URBAN DRAINAGE AREA SIMULATOR (ILLUDAS)

ILLUDAS is an internationally known computer program which has been used for many urban runoff simulations. The characteristic pipe routing method in ILLUDAS is based on the TRRL method. Later, ILLUDAS was developed by Terstriep and Stall (1974). The most important addition to the original TRRL model is the consideration of runoff from permeable surfaces in addition to that from impermeable, the latter contribution being the only part considered in the TRRL model. The pipe routing method is based on kinematic wave theory. As indicated in Figure 2.6 the kinematic wave model is based on that approach to the complete equation of motion, which accounts for only the gravity force and the friction force. It has been shown (Sjoberg 1976) that this approach is not able to take backwater effects into consideration.

The numerical approach to the kinematic wave movement used in ILLUDAS results in wave attenuation, which depends critically on the chosen space step (Δx) (Sjoberg 1976, Smith 1980). Thus the difficulty of using this solution-technique is to determine the appropriate value

of Δx. Engelund and Pedersen (1978) have suggested a method by which the "best" value of Δx can be estimated by considering a triangular input hydrograph.

During pressurised flow the kinematic wave approach cannot be used. When the incoming flow is greater than the full-bore capacity, water accumulates in the upstream manhole until the incoming discharge has decreased below the capacity. This approach is obviously a rough simplification, but was a clear advancement on the previous models.

The characteristics of this simulation program can be outlined as follows:

(i) ILLUDAS pays no attention to the type of flow, i.e. sub-critical or supercritical flow.
(ii) The calculation of friction slope is based on the Manning formula.
(iii) Head losses in manholes and junctions are not explictly accounted for.
(iv) ILLUDAS includes the simulation of special flow regulation devices such as retention basins, weirs and pumps.
(v) Only branched networks can be modelled.
(vi) Input hydrographs may be created by the surface runoff module or may be specified as external hydrographs.

4.6 STORM WATER MANAGEMENT MODEL (SWMM)

Perhaps the most widely known of the computer-based urban rainfall-runoff models is the Storm Water Management Model (SWMM). This model which is similar in form to that embodied in the Chicago Hydrograph Method is available as a program containing over 10,000 FORTRAN statements (Torno 1975). It was originally developed by a consortium of two firms of consulting engineers and a university under contract to the then US Federal Water Quality Administration (now the US Environmental Protection Agency).

SWMM differs from the Chicago Hydrograph Method and its contemporaries in treating both the quality and quantity of urban

runoff. Torno (1975) has described SWMM as consisting of four major blocks of subroutines controlled by a fifth group of executive routines. The RUNOFF block is concerned with the derivation of runoff hydrographs and their associated pollutant loadings. The TRANSPORT block routes both the hydrographs and the time variations of individual pollutants (also referred to as 'pollutographs' or 'chemographs') through the sewerage system. The STORAGE and RECEIV blocks simulate the action of a sewage treatment plant and the impact of discharges on the watercourse receiving the effluent respectively. The water quality models incorporated into SWMM, which have been discussed by Lager et al (1971), are considered more fully by Hall (1984) and Jacobsen (1983). The hydrological model contained within the RUNOFF block was described by Chen and Shubinski (1971). They stated that the overland flow hydrograph from each catchment plane is derived from water balance computations at each step in which allowances are made for both infiltration and depression storage. These overland flow hydrographs are then routed through channel storage. The subsequent routing of flows through lateral sewers may be carried out either in the RUNOFF block using a simplified approach or, where backwater effects are likely to be significant, in the TRANSPORT block using a more sophisticated technique.

The TRANSPORT block is the routine which routes the storm water flows through a conveying branch sewer network. Dry weather flows and infiltration into the sewer system can also be computed. Two types of flow diversion structures, two storage basins and one lift pumping station can be modelled. The model is limited to the simulation of single storm events and to a maximum of 160 hydraulic elements. Larger systems may be handled by modelling the major areas separately. The flow routing is based on the quasi-steady dynamic wave approximation, this being a form of the Saint Venant equations (Eqns 3.1 and 3.2), with the omission of the local acceleration term.

A Newton-Raphson technique is applied to solve the non-linear continuity equation and the friction slope is evaluated from Manning's formula. The solution procedure basically follows a kinematic wave approach in which disturbances are allowed to propagate only in the downstream direction. As a consequence, backwater effects are not modelled beyond the domain of a single conduit and downstream conditions will not affect upstream elements.

However, backwater effects can be approximated at a maximum of two locations by specifying a storage element and providing appropriate geometric input data. The program does not simulate pressurised flow conditions, flows in excess of the full flow conduit capacity being stored at the upstream manhole until conditions permit the accommodation of the volume stored. Only dendritic systems are permitted and inter-connections, looped networks and flow reversal cannot be simulated.

4.7 S - 11 - S

The computer program 'System 11 Sewer' or S11S was developed at the Danish Hydraulic Institute (DHI). It is a development of the original System 11 which is a modelling system for one-dimensional, single-layered i.e. vertically homogeneous flows in natural water bodies. The S11 model was based upon research on one-dimensional flows carried out during the 1960s (Abbott & Ionescu 1967, Abbott & Verhoog 1968).

During 1972 to 1974, it was developed into an operational modelling system with hydrodynamic, transport diffusion and water quality stages. S11S was intended firstly for applications to rivers, estuaries, fjords and similar water bodies. It was then applied extensively from 1974 onwards and hence a considerable body of operational experience and associated system development was accumulated (Abbott & Cunge 1982). The hydrodynamic stage of the S11S used an implicit finite-difference scheme providing second- and higher-order accuracy whilst the double-sweep method was optimized for accuracy, speed and input-output flexibility (Cunge et al 1980).

As the initial version of S11S was developed into a reliable instrument of river engineering practice, the special version for applications to flows in storm water sewer systems was later developed. In 1983, S11S was made available for municipal authorities and private engineering companies at a computer service bureau (Jacobsen 1983). The program has also been included in the Danish Urban Runoff Package called the SVK-system which offers a

number of urban runoff simulation programs together with selected historical rain series.

The below-ground flow routing procedure in the sewer simulation version of S11S is based on the complete Saint-Venant equations which consist of equations of continuity and motion i.e. Equations 3.1 and 3.2. The routing of free surface flow is based on the kinematic wave approximation. In the event that the flow becomes pressurised, the Saint-Venant equations are extended by introducing the 'free-surface analogy' which is also called the 'Preissmann slot method' (DHI 1980, Hoff-Clausen et al 1981). In pressurised conditions the flow is allowed in the imaginary slot above the conduit (Figure 4.1) so that the same routing method can be kept throughout both free-surface and pressurised flows (Preissmann & Cunge 1961, Cunge & Wegner 1964). Similar to the initial river version, the Saint-Venant equations are represented by the implicit finite-difference scheme and is solved by a double sweep algorithm.

S11S is able to take into account backwater effects and to simulate surcharged systems. Other characteristics of the program are such as :

(i) Standing and moving hydraulic jumps are automatically accommodated,
(ii) Calculation of friction slope is based on the Manning formula,
(iii) Head losses in manholes and junctions are accounted for with the coefficient as an input parameter,
(iv) Weirs may be located at manholes and are described as standard broadcrested weirs,
(v) Dendritic and looped systems are able to be simulated by the program.

4.8 WALLINGFORD PROCEDURE STORM SIMULATION PACKAGE (WASSP)

The Wallingford Procedure for the design and analysis of urban storm drainage networks is based upon the results of a collaborative research programme carried out in the United Kingdom between 1974 and 1981 by the Hydraulics Research Station, the Institute of Hydrology

and the Meteorological Office. The work was coordinated by the National Water Council / Department of the Environment Working Party on the Hydraulic Design of Storm Sewers. The Procedure consists of four methods:

1. The Rational Method
   A modified version of the Rational Method intended for use on outline designs on homogeneous areas of up to 150 ha; both manual and computer-based versions of this method are available the latter including the facility to model stormwater overflows.
2. The Hydrograph Method
   A computer-based approach which models the above-ground and below-ground phases of runoff separately. This method may be employed for both design and simulation (Price & Kidd 1978), and allowances may also be made for the action of stormwater overflows, on-line and off-line detention tanks and pumping stations.
3. The Optimising Method
   A computer-based technique for obtaining the pipe diameter, depth and gradient associated with the minimum construction cost using the discrete differential dynamic programming technique (Mays & Yen 1975, Price 1978[2]).
4. The Simulation Program
   A computer-based method with which the performance of both an existing system and a proposed design may be examined under surcharged and free surface conditions (Bettess et al 1978). Ancilliaries such as stormwater overflows, on- and off-line detention tanks and pumping stations may also be taken into account.

These methods may be applied to both separate and combined sewerage systems, although the calculation of foul sewage flows is not included. No allowances are made for the calculation of runoff from any rural areas that may contribute to the urban drainage network, and no water quality modelling is attempted at present. Nevertheless, the Procedure allows the hydraulic and cost consequences of alternative design standards relating both to the pipe network and any ancillary structures to be evaluated for a minimum expenditure of time and effort on the part of the designer.

The selection of the method most appropriate for a particular design requirement is assisted by following the flowchart presented in Figure 4.2. For the design of new systems, any of the Modified Rational, Hydrograph or Optimising Methods can be employed. The flow calculations for the Optimising Method are carried out using the Modified Rational Method, and so the discharge estimates obtained from these approaches should be similar, unless the gradient optimisation substantially alters times of concentration.

For the analysis of an existing system, the Modified Rational, the Hydrograph Methods or the Simulation Program may all be used. The Modified Rational Method is, as before, limited to the estimation of peak flow rates. The Simulation program incorporates the same algorithm for simulating the above-ground phase of runoff as the Hydrograph Method. The pipe-routing technique is also the same until surcharging begins, and so both of these methods should yield similar results in non-surcharged pipe systems.

For both the design of new systems and the simulation of existing sewer networks, different methods may be more appropriate at different stages of an investigation. The Modified Rational Method may be applied for both design and analysis in order to provide an initial appreciation of catchment response. For a new sewerage system, the Optimising Method might then be employed to determine pipe, sizes, depths and gradients, which subsequently can be checked using the Hydrograph Method. The latter approch can also be applied to check an existing system for surcharging. Finally, the Simulation program both allows the performance of a proposed sewer network to be evaluated when subjected to rarer events than the selected design storm, and permits a more detailed examination of zones of surcharging in an existing pipe system.

One particular aspect of the WASSP package which has been used extensively in this research is the sewered sub-area model. This has been developed (Packman et al 1981, Nussey 1986) for situations where insufficient data are available to permit the modelling of both the above-ground and the below-ground phases of runoff for every subcatchment and pipe length, or where the costs of data collection for a large drainage area would be prohibitive, a simplified sub-area model is available in WASSP. In this model, the method of computing

the gulley hydrographs is applied to sub-areas of up to 60 ha instead of each pipe length. As shown schematically in Fig. 4.3, the computer sub-area hydrograph is then divided into N equal parts and distributed equally to the N segments of an 'equivalent pipe'. The latter consists of a tapered system of pipes in series, each of which has the same length and slope. The number of segments, N, depends upon the time of flow within the equivalent pipe.

The model requires as input data the total length of the major pipe run in the sub-area, the average pipe slope, and the diameter and slope of the outfall pipe. Where no details of the outflow pipe are available, as in a design application, its dimensions must be estimated using the Modified Rational Method. Using this Sewered Sub-area Model, substantial savings on input data are possible, with networks of the order of 100 pipes being reduced to only four equivalent pipes (Kidd & Packman 1980). As before, routing of flows through the equivalent pipes is carried out using the Muskingum-Cunge Method. However, over-estimation in the runoff hydrographs is often the case due to the simple input data and over-simplistic treatment of the catchment wetness prior to events (Packman 1986). In addition it cannot be used on a system where surcharging is possible. The Sewered Sub-Area model cannot be adopted for every sewer network. More research needs to be carried out so that SSA can be applied to any system effectively (Ashley et al 1986).

4.9 DUNDEE COLLEGE OF TECHNOLOGY SEWERAGE MODEL (DUCTS)

An investigation into the performance of The Dunfermline Sewerage System (described in Chapter 5) by Dundee College of Technology was begun in 1983 in order to model the performance of the sewerage system (Ashley & Jefferies 1983). Since then a simulation model called DUCTS has been set up (Ashley & Jefferies 1984) and applied to Dunfermline. DUCTS was developed using the published documents of Wallingford Procedure (HRS 1981) and IOH reports (1974-1979), and the model is as a result, reasonably close to the WASSP. In the model development stage, only design storms were used together with the real catchment data of Dunfermline.

Between 1984 and 1985, the performance of DUCTS was investigated extensively. This primitive version of DUCTS had been tested by using real catchment data with design events (Angus 1985) in order to justify the sophistication of the assembled model and the errors due to the input data and from the computer model itself. Consequence of this investigation by Angus resulted in an improved version of DUCTS. However, the model still lacked the process of calibration and verification by means of some observed events and corresponding discharge hydrographs.

This improved version of DUCTS was later used on the self-contained sewered sub-catchments in Dunfermline and simulation outputs had been compared with both observed data and with the commercial package WASSP (see Chapter 7 for details).

The required input data for DUCTS include:

(i) physical system definition data (SSD), and
(ii) rainfall-flow data (PCD), which indicate the system response characteristics.

DUCTS, similar to the Wallingford Procedure, employs a sewered sub-area model. Those sub-catchments which are represented by the SSA model have to be input in the SSD file as indicated by a flag number. SSA models have to be input at the top of a new branch because DUCTS does not recognise SSA at the pipe branch due to the unique manhole downstream number computation procedure. Despite this limitation, the performance of DUCTS was considered to be satisfactory when compared with the real flow data and the simulation results by WASSP (see Chapter 7). A flow chart of DUCTS is shown in Figure 4.4 indicating the computation procedures.

In simulating the free surface flow in pipes, DUCTS utilises the Muskingum-Cunge method which was shown to be economic in computer time and produced reliable results. In the surcharged condition, DUCTS identifies a group of inter-connected surcharged pipes to form a surcharged sub-system, which is the same procedure as used in WASSP. Pressurised flows are then solved for the whole sub-system.

Mathematical equations and fuller details in dealing with the surcharged flow have been described in Section 2.4.2.

4.10 THE NEED FOR DUPPERS

Urban storm drainage runoff simulation models exist worldwide and there are far too many to be considered in this Chapter. However, the chosen models reviewed range from simple simulators such as the TRRL Method to sophisticated models such as SWMM and WASSP. One common aspect in the development of these models is that they have by and large been generated initially for some specific study catchments. Some later models were further enhanced but are still largely based on the earlier methods. The commercial packages such as WASSP, can only be applied to typical systems, for example dendritic systems.

Parallel pipe systems linked by a number of cross-connections are uncommon. It suggested that parallel pipe systems such as that in Dunfermline could only be modelled by simplifying or modifying the system artificially. However, these models could only predict combined flows at the catchment outfalls (see Chapter 7). No simulators exist which could handle the twin pipe systems without alterations. A commercial version of SPIDA (Osborne 1985) which has a looping facility is apparently to be released in the imminent future. This may not, however, be usable for the Lyneburn parallel pipe system due to the presence of the cross-connections.

Besides the prediction of flows in the parallel pipes, the flow behaviour in the cross-connections had also to be identified and modelled for the Lyneburn system. In order to simulate the complicated parallel pipe system with cross-connections such as the Lyneburn drainage system, an in-house model was required making the case for the parallel pipe model DUPPERS.

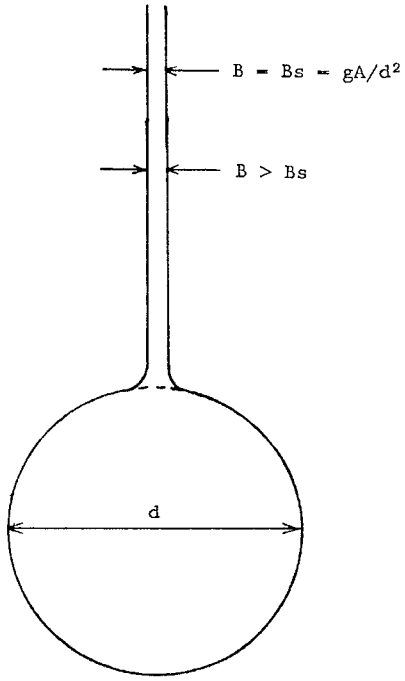


FIGURE 4.1 PREISSMANN SLOT -- CIRCULAR PIPE WITH FICTITIOUS SLOT

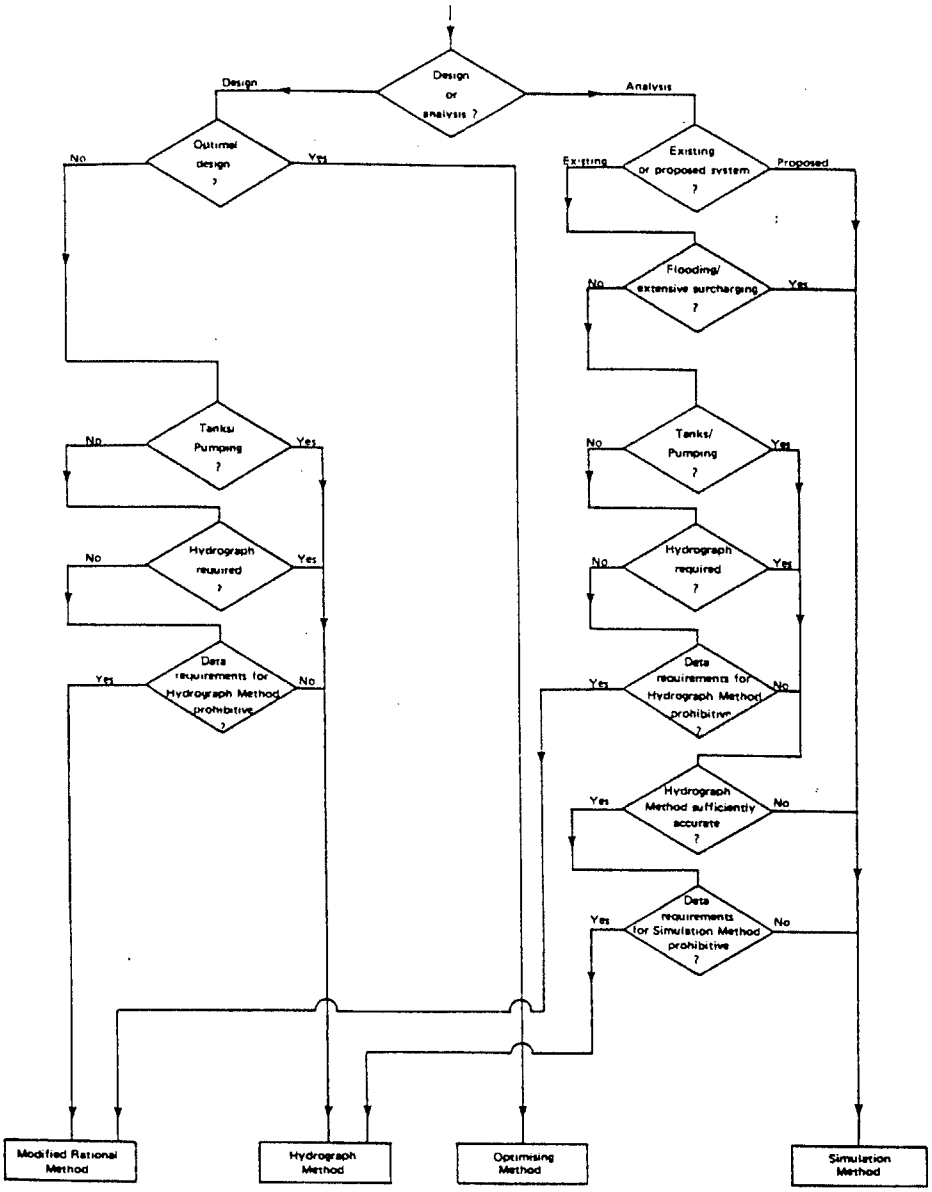


FIGURE 4.2 SELECTION OF THE APPROPRIATE METHOD IN WASSP PACKAGE (HRS 1981)

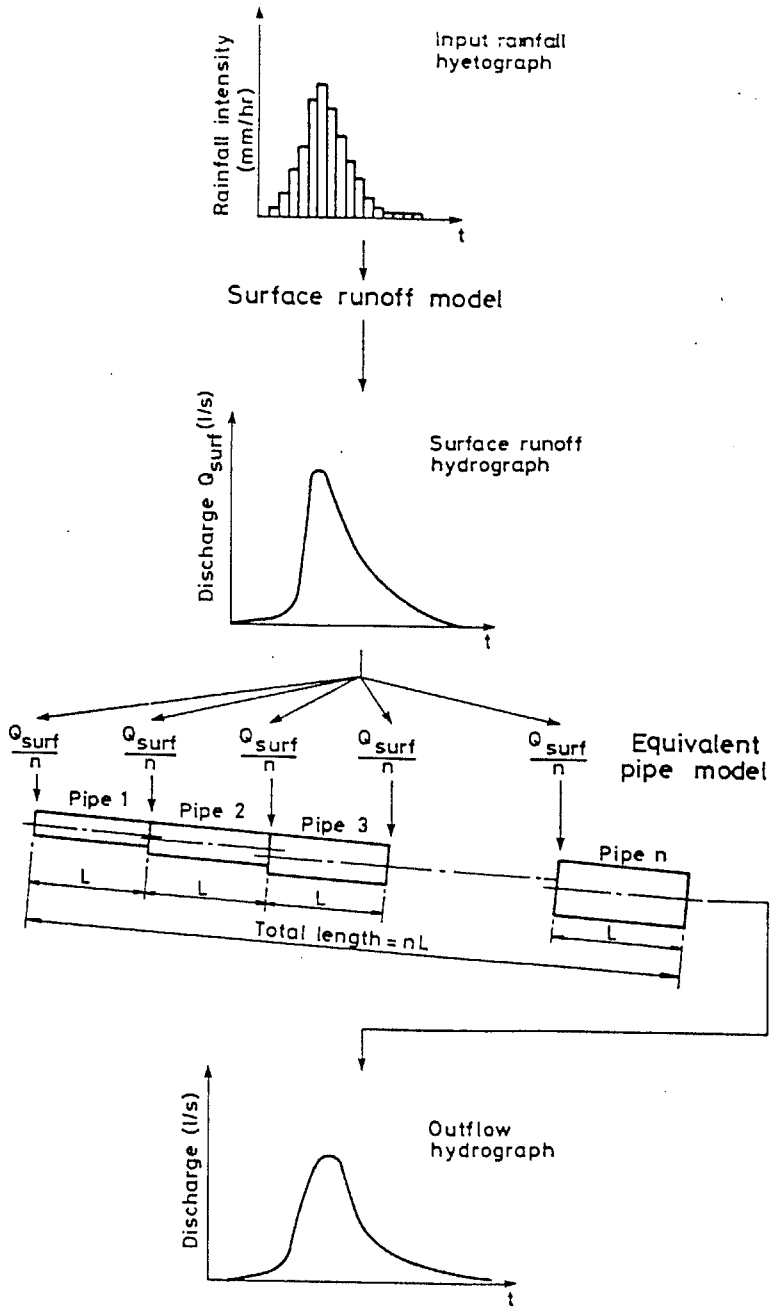


FIGURE 4.3 SEWERED SUB-AREA MODEL IN WALLINGFORD PROCEDURE (HRS 1981)

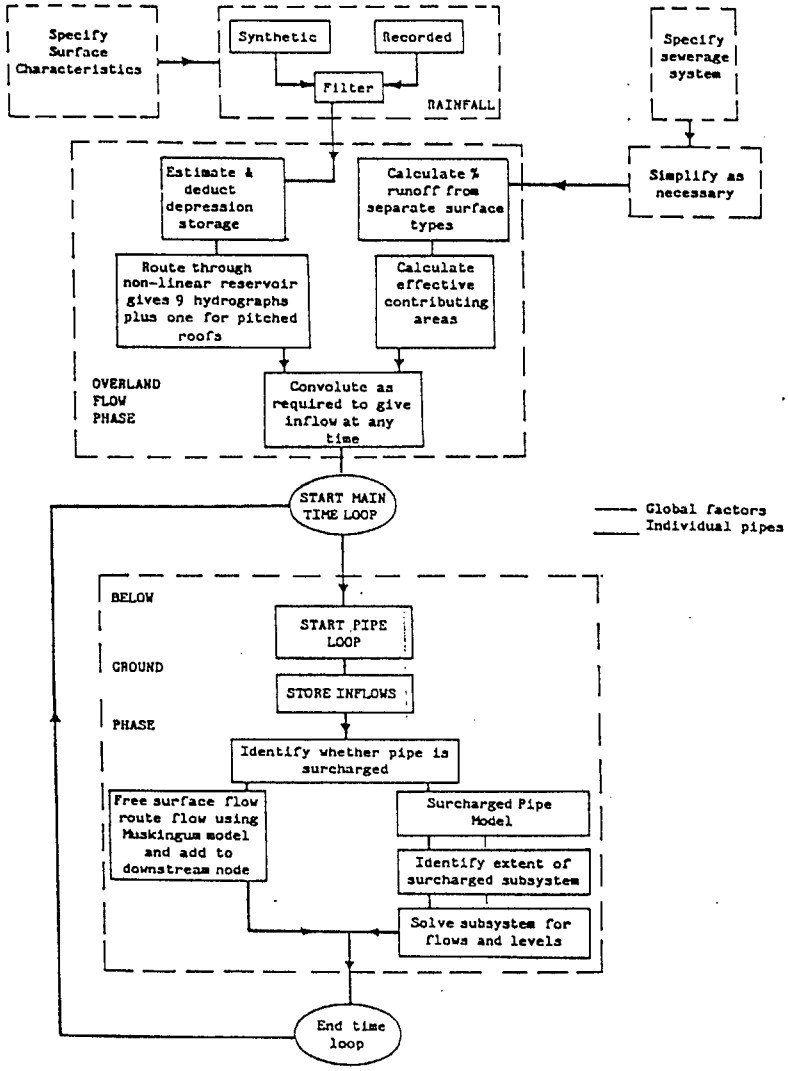


FIGURE 4.4 CONCEPTUAL FLOWCHART FOR DUCTS SEWERAGE SIMULATION MODEL

CHAPTER 5
STUDY CATCHMENT

5.1 INTRODUCTION

Dunfermline, in an area well known for the coal mining in the past (Parry 1983), is located in the centre of Fife. Figure 5.1 shows the location of Dunfermline whilst Figure 5.2 shows the Lyne Burn surface water catchment which includes virtually all of Dunfermline including the Keirsbeath Mine to the north-east of the burgh. The Lyne Burn is a small watercourse which drains an area around Dunfermline, its catchment area being heavily developed with both urban and industrial areas. The burn, apart from conveying unnaturally low flows because of discharge into abandoned mines (Wilson (unknown), Jefferies et al 1985), is fairly typical of urban streams, having moderate pollution and occasional flooding problems. Plate 5.1 shows the normal dry weather flow in the Lyne Burn at Rex Park with bricks, shopping trolleys and other refuse in the burn causing flow restrictions. Bank-full flow caused by a moderate event is shown in Plate 5.2 at about the same location in Rex Park.

Recent urban development has substantially increased the size of Dunfermline and this growth has progressed ever more rapidly since the opening of the Forth Road Bridge (Jefferies and Ashley 1983, Ashley et al 1986). This growth has tended to be in the form of successive peripheral developments spreading out from the older central area. To meet the drainage requirements of the town, the sewer system has been continually added to, but only on the basis of providing sewerage for the more recent developments. Each additional area has been connected into an ageing trunk sewer system which is at present unable to carry the increased storm flows without extensive surcharging (ibid). The sewer system is very mixed with most older areas combined, however, most recently developed areas are separate systems which drain directly or indirectly to the Lyne Burn (Au-Yeung et al 1986). Plate 5.3 shows an storm relief outfall from a modern housing estate discharging combined flow into the burn during storms.

5.2 LYNEBURN SEWERAGE SYSTEM

For a considerable proportion of its length, the main sewer system follows the Lyne Burn watercourse. The Lyneburn sewers are composed of two parallel, inter-connected pipes nominally for separate foul and for storm flows. The original combined sewer was duplicated around the 1960s and the pipes are inter-connected by approximately 20 overflows and cross-connections in a haphazard, unconventional manner.

There are a total of five main branches which serve the entire Dunfermline catchment :

(i) Lyneburn,
(ii) Calaisburn,
(iii) Central Park,
(iv) City Centre, and
(v) Towerburn.

The locations and layout of the above main sewers are shown in Figure 5.3. For the research study, an overflow chamber at Bothwell Street (Figure 5.4) was chosen to be the lower limit of the system for a model of the parallel pipe system and hence only the first four of the above main branches required to be considered. The Bothwell Street chamber is the last cross-connection overflow along the Lyneburn parallel pipe system. After this chamber, the foul sewage discharges to the treatment works at St. Margaret's Bay whilst the storm flow is via the Dunfermline storm relief sewer discharging to the Lyne Burn at Waukmill (Jefferies et al 1986).

In addition to the main branches given above, four subsidiary areas at the periphery of the system are of note. It has been found that these areas contribute little to the flows downstream, either because they are very remote, or because their sewers consist of separate systems (Au-Yeung 1986).

These areas are :

(i) Crossgates -- remote and separate system,
(ii) Halbeath -- separate system,
(iii) Kingseat -- remote area,
(iv) Bellyeoman -- remote area.

The locations of these subsidiary branches are shown in Figure 5.4.

5.3 CATCHMENT SURVEY

Very often the catchment charateristics and the system require to be investigated thoroughly for the purposes of either design of new sewer systems or for sewer renovation. The hydraulic design of new sewer systems has been carried out based on the catchment size and population and subsequent to determine the size of sewers for a given layout (Eadon 1986). Rehabilitation design, however, has to deal with hydraulic and structural problems which already exist. Before simulation of an existing system, an overland survey is required for the model construction (Williams 1984, Eadon 1984, Moss 1985). An underground sewer survey, on the other hand, provides information such as hydraulic performance of sewers (Watts 1986) and locates the 'critical' sewers (Read 1984, Williams & Bartlett 1984). Sometimes underground sewer surveys can also identify locations where infiltration is extensive (Martin et al 1982).

Standard procedures on the overland survey and underground sewer investigation have been outlined in detail in the Sewerage Rehabilitation Manual (WRc/WAA 1984) and Sewers For Adoption (WRc/WAA 1981 &1985). Above ground survey is considered to be the necessary stage for the model calibration of an existing system (Eadon 1984). The subsequent accuracy of the model is directly dependent upon the quality of the above ground survey information. It is also important to define the sub-areas as 'separate', 'partially separate' or 'combined' systems, since erroneous inclusion of contributing area is likely to cause serious over-

prediction by the model. More detailed overland survey may be required for model verification. It is good practice to have appropriate sizes and scales of maps available to define sub-catchments (Styles & Robinson 1984) and record the above ground survey information (Styles & Hedderly 1982).

The condition of sewers requires to be examined regularly because of the effect on the hydraulic performance of the system. They have to be maintained frequently and free from collapse due to age or failed materials. The Sewerage Rehabilitation Manual lists the classification of sewers to be inspected and the duration of surveys. Recently in-sewer closed circuit television (CCTV) has become available (Moss 1985, Robson 1986) and hence inspection in smaller and non-man entry sewers has been made easy. CCTV has become increasingly popular since its launch in the 1960s and is used mainly on those areas where the cost of collapse repair is highest and also for critical sewers (Fiddes 1984). Sometimes coloured dye is also used to trace 'missing' sewers.

5.3.1 ABOVE GROUND SURVEY

The main purpose of carrying out above ground surveys for the Lyneburn catchment was to identify the contributing areas and to define the characteristics of the sub-catchments. In general the sewer records were adequate for the routes of sewers, although they were not updated sufficiently and several modern housing estates and recently redeveloped areas were not thus recorded on these plans. The record plans, scaled 1:1250 and based on Ordnance Survey Maps, were only used for the determination of pipe lengths and the measurement of contributing areas.

Due to the size of the study catchment and to make efficient use of the limited time available, typical sub-catchments were chosen for detailed overland surveys by virtually 'walking' over the whole sub-catchment. Two sub-catchments, Scotland Drive and Garvock Bank (Figure 5.5), were surveyed in detail to establish reliably such

parameters as :

(i) Overall contributing area to each pipe (AREAC),
(ii) Percentage of impermeable area and pitch roofs (PIMP and PRroof respectively),
(iii) Paved area per gulley (PAPG),
(iv) Sub-catchment gradient (SLOPE).

Pipe invert levels and cover levels were mainly taken from the sewer plans. Additional levelling, however, was required for most of the main parallel sewers due to the missing and unreliable records.

Simplified and full models were then constructed for the two surveyed sub-catchments (Angus 1984, Au-Yeung 1986). The survey information, simplification process and subsequent simulation outputs have been summarised and presented in Chapter 7. The values derived for the relevant catchment parameters were then applied appropriately over the full catchment thus supplying reasonably consistent and, hopefully, reliable values while at the same time reducing the survey effort required. The catchment parameters obtained are summarised in Table 5.1 against the relevant housing type in the particular sub-catchments.

The overland survey was continued on-foot for the rest of the study catchment but only to the extent of identification of sub-area boundaries according to the topographical features and the sewer records. The nature of the sub-areas was also determined in order to identify their system type, i.e. combined, partially separate or separate system and housing type. Figure 5.6 shows the sub-areas following this survey work.

5.3.2 BELOW GROUND SEWER SURVEY

In any sewer simulation study it is advisable to carry out some below ground survey work to confirm the details shown on the record plans. The sewer records for the Lyneburn system are out of date and far from trustworthy. A section of pipe records for the

Calaisburn Sewer is completely missing and the locations of manholes had been altered due to housing development. Frequent surface flooding and severe foul discharge into the Burns had been reported from the local residents along the main sewers (Jefferies et al 1985). Figure 5.5 shows these sewers subjected to the frequent surface flooding along the parallel pipe system and many of these are actually the locations of cross-connection overflows. The sewer system is considered to be unsatisfactory for the following reasons :

(i) During heavy rainfall both pipes of the parallel system become completely surcharged and a proportion of the flow is above ground after manhole covers have been blown. This occurred once during the research period on 17 June 1986, an event with a rainfall return period of five years. Maximum water levels during this event are shown in Figure 5.7.

(ii) There are intermittent discharges from the sewers to the watercourses at various locations during times of high flows.

(iii) Frequently blockages occur causing the relief sewer to carry foul sewage. This results in unsatisfactory conditions at the outfall at Ironmill Bay.

(iv) Combined manholes with foul and storm drains separated by a simple weir enables foul flow to enter the storm relief pipe. Same consequence as (iii) above results.

(v) Wrong surface connections into the foul sewers exist contributing unnecessarily high flows to the Dunfermline sewage treatment works.

An extended sewer survey was carried out to check on pipe diameters, connections at manholes and interconnections on the parallel pipe system. The physical sizes of the cross-connections and the overflow structures were also recorded since these were not available. Additional sewer information such as depth of silt, pipe materials and roughness were also recorded for the model construction. The records were booked using standard recording forms as set by the Standing Technical Committee (NWC/DoE 1980).

An example of a manhole survey is shown in Appendix F. Virtually all main sewers in Figure 5.3 were surveyed in approximately four months during the summer of 1985.

5.4 CHARACTERISTICS OF LYNEBURN DRAINAGE SYSTEM

The Lyneburn sewerage system serves a population of 52,000 and has four main branches. The full system to the Bothwell Street chamber contains a total of approximately 1500 discrete sewers, the largest of which is 1800 mm in diameter.

The trunk Lyneburn sewerage system consists of parallel pipes with overflows and cross-connections installed at random locations. These main parallel sewers drain some 650 ha of the total 1100 ha of sewered catchment leading to the Bothwell Street chamber. The twin sewers are nominally for separate storm and foul flows but they are actually combined and re-separated via the overflows and cross-connections.

5.4.1 CONTRIBUTING AREAS

The contributing areas have been defined for all the main sewers shown in Figure 5.3. The catchment boundaries for all sewers were sketched on the 1:1250 sewer plans and verified by overland surveys. The catchment boundaries for the main branches are shown in Figure 5.8.

Overland characteristics and the average gradient for the other sub-catchments and global catchment were also recorded during the above ground survey. The percentage of impermable area (PIMP) and pitched roofs (PRroof) as shown in Table 5.1 indicate that the City Centre Sewer has the highest paved and roofed areas while the 1960s and 1970s housing estate such as Scotland Drive bear the lowest paved areas due to the high amounts of gardens and public open space.

The contributing areas and the corresponding sewer lengths for all the four main sewers are recorded and summarised in Table 5.2.

5.4.2 PARALLEL PIPES

Besides the main trunk Lyneburn Sewer, part of the secondary branches of the network including the Calaisburn and Bellyeoman Sewers are twin pipe system but without the presence of the cross-connections or overflows. The Park Sewer, however, is also a parallel pipe system but both pipes meet in combined manholes enabling the foul and storm flows to be mixed during surcharge. The two outfalls of the Park Sewer join and mix in a manhole before the flow enters the main Lyneburn Sewer and hence the Park Sewer can be considered as a conventional combined system.

All inflows from the contributing sub-areas to the Lyneburn main sewer discharge directly to the foul pipe of the parallel sewers. Hence in normal dry weather conditions, only the foul pipe of the parallel sewers conveys flow and there is no flow in the storm relief pipe as far downstream as the Mill Road flow monitoring site. However, a small dry weather flow is present in the storm pipe at Bothwell Street chamber due to minor infiltrations and a small 'cundie' i.e. stream which has been directed into this pipe. This connection is in the mid section of the Park Sewer.

The Lyneburn parallel sewers start where the Halbeath and Bellyeoman Sewers join at Halbeath Drive (Figure 5.4). At Woodmill Road, flows in the two pipes join together and enter an ovoid tunnel. The parallel pipes resume again at the downstream end of the tunnel, overflow from foul to storm relief pipes is possible here during high flows. Since flows combine at the tunnel, for modelling considerations the twin pipes are lumped into a single equivalent pipe above this point. The methodology of combining the twin pipes into a single equivalent pipe has been shown in Chapter 7 and Appendix D.

In order to have a better understanding of the hydraulic behaviour of the twin pipes and to construct a parallel pipe model, a longitudinal profile was drawn for the section of parallel pipes

from the downstream end of the tunnel to Bothwell Street overflow chamber. The profile is included as Appendix E. This appendix also includes the hydraulic characteristics of the twin pipes and other details.

5.4.3. CROSS-CONNECTION OVERFLOWS

Since publication of the first edition of the Sewerage Rehabilitation Manual (WRc/WAA 1983), approaches towards sewer upgrading and renovation have been revised to incorporate significant advances which have taken place in a number of research areas (Rofe 1986) of particular note are procedures for avoiding pollution (Clifforde & Price 1986, Henderson & Moys 1987, Saul 1988[1], Jefferies & Stevens 1989); in structural assessment of sewers (Fiddes 1984, Clifforde 1984, Williams & Bartlett 1984, Moss 1985); and in renovation techniques (Read 1982 & 1984). Recently real-time control has become more popular worldwide (Beron et al 1984, Schilling 1984, Smisson 1987, Harding 1986). The use of hydraulic structures in sewerage systems has increased. These include storage tanks (Taylor & Knott 1986, Saul 1989), storm sewage overflows (Chaplin 1985, Saul & Murrell 1986) and hydro-brakes (Pratt & Balmforth 1986, HR & D 1987). Techniques for the reduction of overland flows into sewerage systems are also widely available using infiltration and soakaways (Pratt 1986, Pratt & Balmforth 1986, Pratt 1987, Watkins 1987).

Combined sewer overflows (SSO), such as those in the Lyneburn parallel sewerage system, are frequently used as flood alleviation devices on British sewerage schemes. The primary function of this type of overflow is to prevent flooding and to restrict the volume of throughflow to treatment. The excess sewage flow above the setting is discharged either to the nearest watercourse or to a second sewer system which has available capacity. It is also anticipated that the overflow should achieve solids separation and retention on the first foul flush (Saul & Thornton 1989). The design recommendations for an overflow has been presented in Technical Committee report (HMSO 1970) and in the recently published WRc report (Balmforth & Henderson 1988).

Most of the overflows in the parallel pipe system under consideration are low side weirs, i.e. height of the weir is less than half the diameter of the inlet pipe. This type of overflow weir was frequently used in British practice with either single or double side weirs and has been shown to have both poor hydraulic characteristics and solid separation performance (Ackers et al 1967). It has been suggested (Saul & Murrell 1986) that all such overflows should be modified by increasing the height of the weir to form a high-sided overflow and to incorporate some form of additional throttle control.

Most of the overflows on the Lyneburn system are found in the upstream part of the parallel pipes before the ovoid tunnel. There are six cross-connections and overflows located after the tunnel including the Bothwell Street overflow chamber. The downstream section of the Lyneburn parallel pipes and the locations of these cross-connections are shown in Figure 5.9. Plates 5.4 and 5.5 show the locations of the parallel pipes at a cross-connection (No. 4 in Figure 5.9) in Rex Park. It can be seen from the plates that the location here has a very mild catchment slope and the distance between the twin pipes is very small.

Two typical types of overflows and cross-connections can be classified for the overflows in the study system as shown in Figure 5.10 and all are low side weir types. The single side weir overflow, as shown in Figure 5.10(a), is located at the invert level in the manhole with the bridging pipe adjacent to the weir. The double side weir overflow, as shown in Figure 5.10(b), is situated in the middle of the manhole. The bridging pipe in this case is located in the invert of the manhole at a lower level than the foul pipe. Discharges over the weirs for the two cases are carried over to the storm relief pipe via the bridging pipes. The lower part of the chamber in type (b) does allow some additional storage during surcharge. Plate 5.6 shows overflow taking place during a storm on 1/4/85 in the Bothwell Street chamber which has double low side weirs. Table 5.3 (a & b) summarises the hydraulic characteristics and levels for the five cross-connections corresponding to those shown in Figure 5.9.

SUB-CATCHMENT TYPE	PERCENTAGE OF IMPERMEABLE AREA (PIMP) (%)	PERCENTAGE OF ROOFED AREA (PRroof) (%)	PAVED AREA PER GULLEY (PAPG) (WASSP only)
Council Housing	58	11	-2†
Estate (60s and 70s)	14	10	-1
City Centre	63	15	-1
Victorian Suburban	26	10	-3
Light Industry	60	8	-2

† Index for PAPG (HRS 1981) :
-1 Less than 200 m^2
-2 Between 200 and 400 m^2
-3 Greater than 400 m^2

TABLE 5.1 Summary of Sub-catchment Type and Their Catchment Characteristics.

MAIN SEWER	SEWER LENGTH (km)	CONTRIBUTING AREA (ha)
Lyneburn	3.2	326
Calaisburn	1.9	157
Park Sewer	1.7	177
City Centre	0.3	15
Bothwell Street	/	(total) 675

TABLE 5.2 Lengths and Size of Contributing Areas for the Study Catchment Main Sewers

CROSS CONNECTION REFERENCE NUMBER	UPSTREAM PIPE				DOWNSTREAM PIPE				MANHOLE PLAN AREA	
	FOUL DIA (mm)	INVERT LEVEL (m AOD)	STORM DIA (mm)	INVERT LEVEL (M AOD)	FOUL DIA (mm)	INVERT LEVEL (m AOD)	STORM DIA (mm)	INVERT LEVEL (m AOD)	FOUL (m^2)	STORM (m^2)
1	600	60.789	/	/	450	60.643	800	59.793	2.10	2.10
2	450	55.770	975	54.062	450	55.590	975	53.940	1.50	1.50
3	375	52.880	1050	50.456	375	52.670	1050	50.315	3.40	2.16
4	375	45.915	1050	44.657	450	45.875	1050	44.455	1.65	9.50
5	375	44.472	1140	43.475	375	44.325	1140	43.340	5.05	2.45
Bothwell St Chamber	600x900 (egg)	41.080	1200	39.602	800x1200 (egg)	40.690	1800	39.170	4.00 x Chamber Size	8.21

(a) Parallel Pipe Details

CROSS CONNECTION REFERENCE NUMBER	BRIDGING PIPE			OVERFLOW WEIR			
	DIAMETER (mm)	LENGTH (m)	INLET INV LEV (m AOD)	TYPE *	DEPTH (mm)	LENGTH (m)	CREST LEV (m AOD)
1	700	3.0	60.643	I	350	2.0	61.139
2	450	2.0	55.590	I	350	1.2	56.120
3	450	4.0	51.215	II	150	2.0	53.030
4	450	3.0	45.875	I	375	1.5	46.290
5	450	3.0	43.570	II	175	2.5	44.647
Bothwell St Chamber	/	/	/	II	300	8.2	41.380

(b) Bridging Pipe and Overweir Details

* Overflow Type (see Figure 5.10) : I -- Single Weir Overflow [Figure 5.10(a)]
II -- Double Weir Overflow [Figure 5.10(b)]

TABLE 5.3 Hydraulic Characteristics and Weir Dimensions for the Five Cross-Connections and Overflows Shown in Figure 5.9

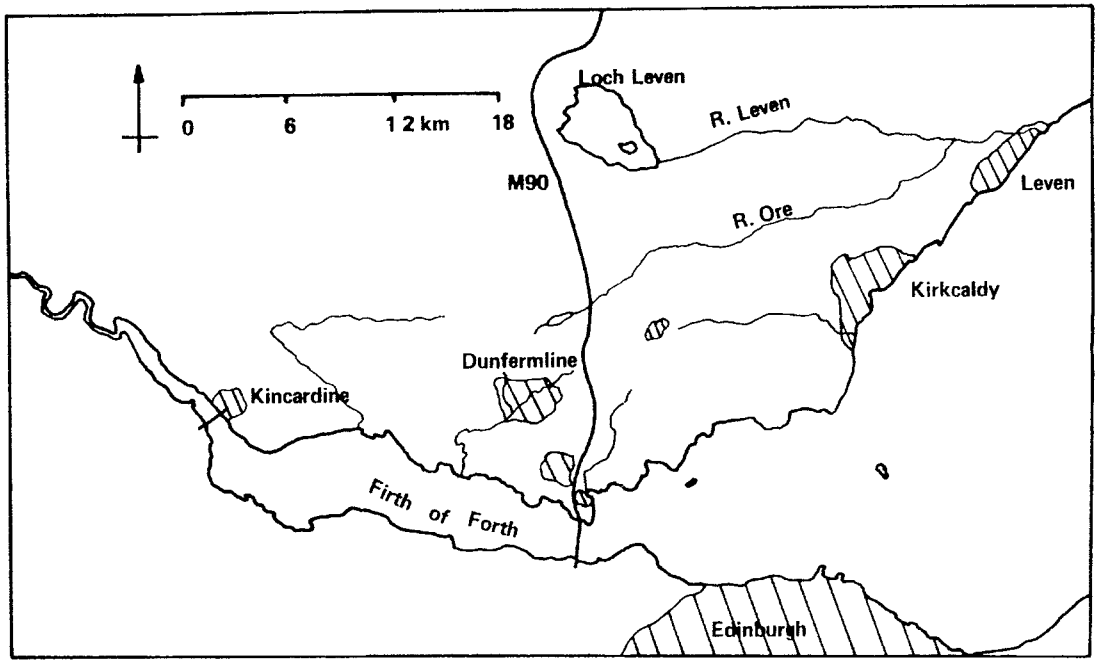


FIGURE 5.1 Location of Dunfermline in Fife Region

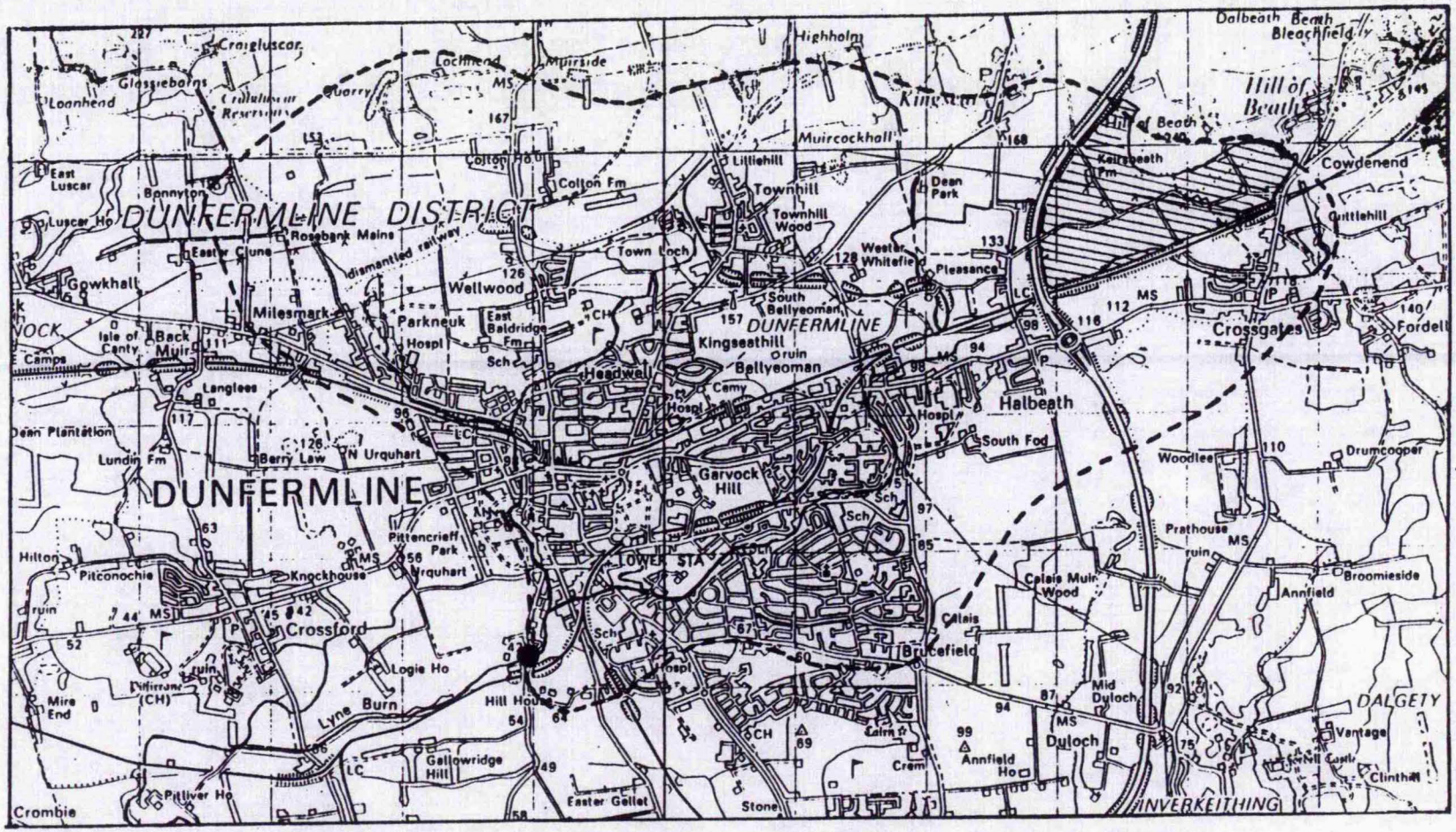


FIGURE 5.2 LYNEBURN CATCHMENT AT LIGGAR'S BRIDGE (Scale 1:50000)
Based upon the Ordinance Survey Map with the permission of the Controller of HMSO, Crown copyright reserved.

Keirsbeath Mine ● Liggar's Bridge

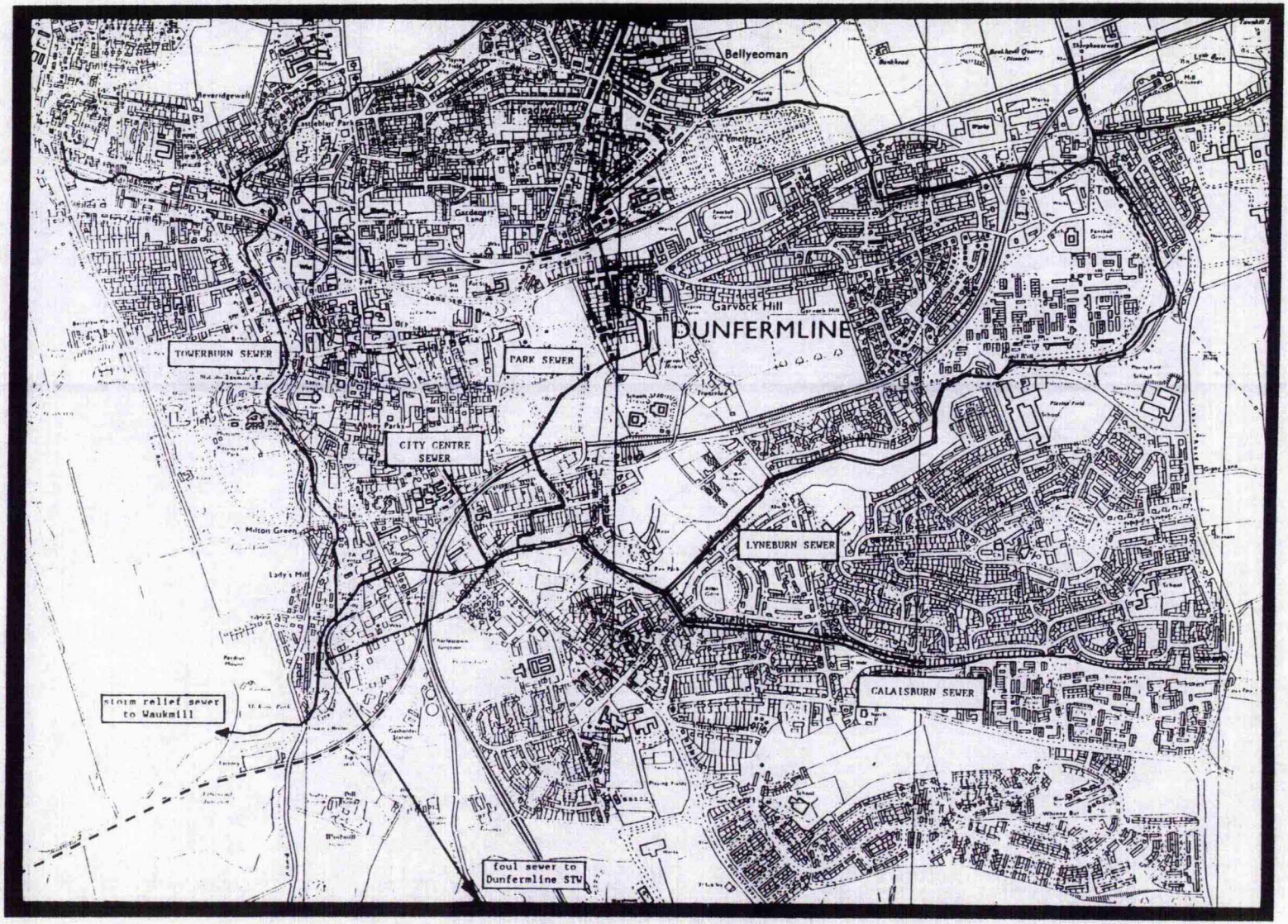


FIGURE 5.3 Locations of the Five Main Sewers in Dunfermline

Based upon the Ordinance Survey Map with the permission of the Controller of HMSO, Crown copyright reserved.

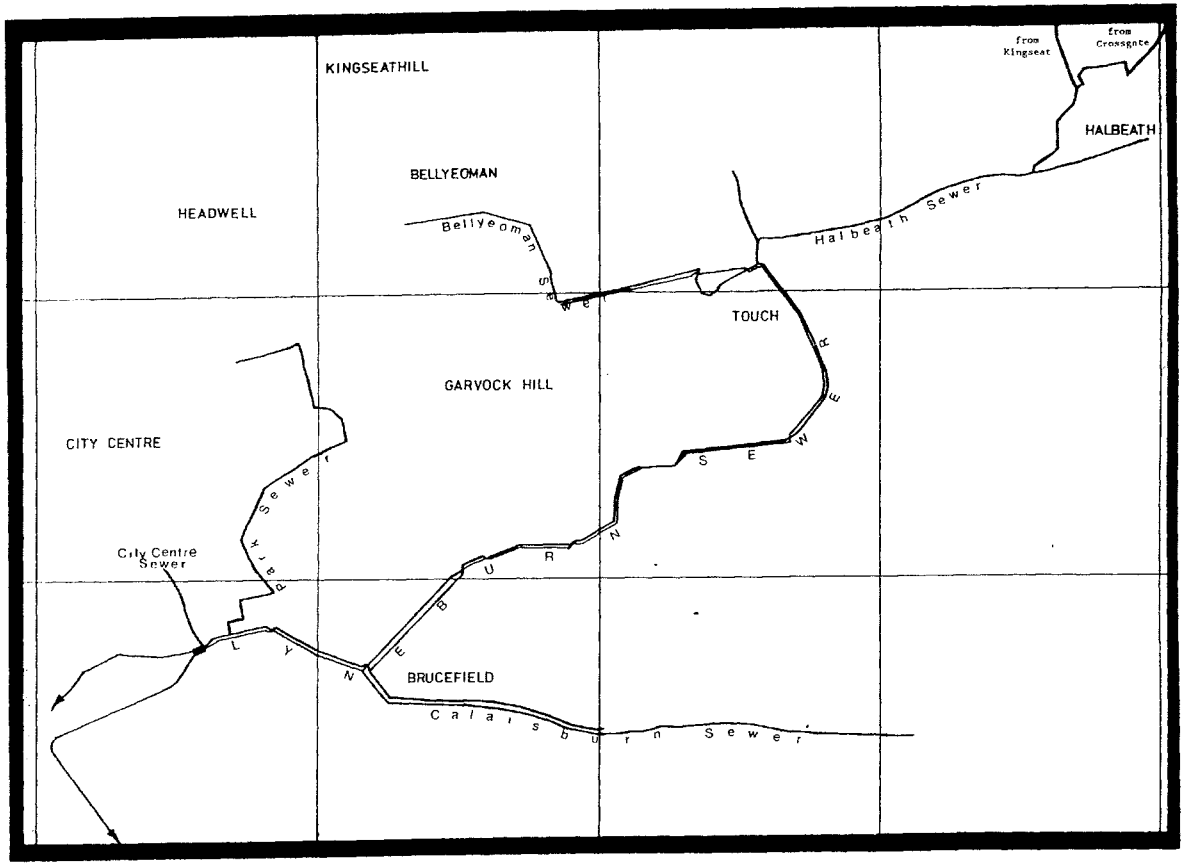


FIGURE 5.4 MAIN AND SUBSIDIARY SEWERS IN LYNEBURN STUDY CATCHMENT

■ Bothwell Street Overflow Chamber

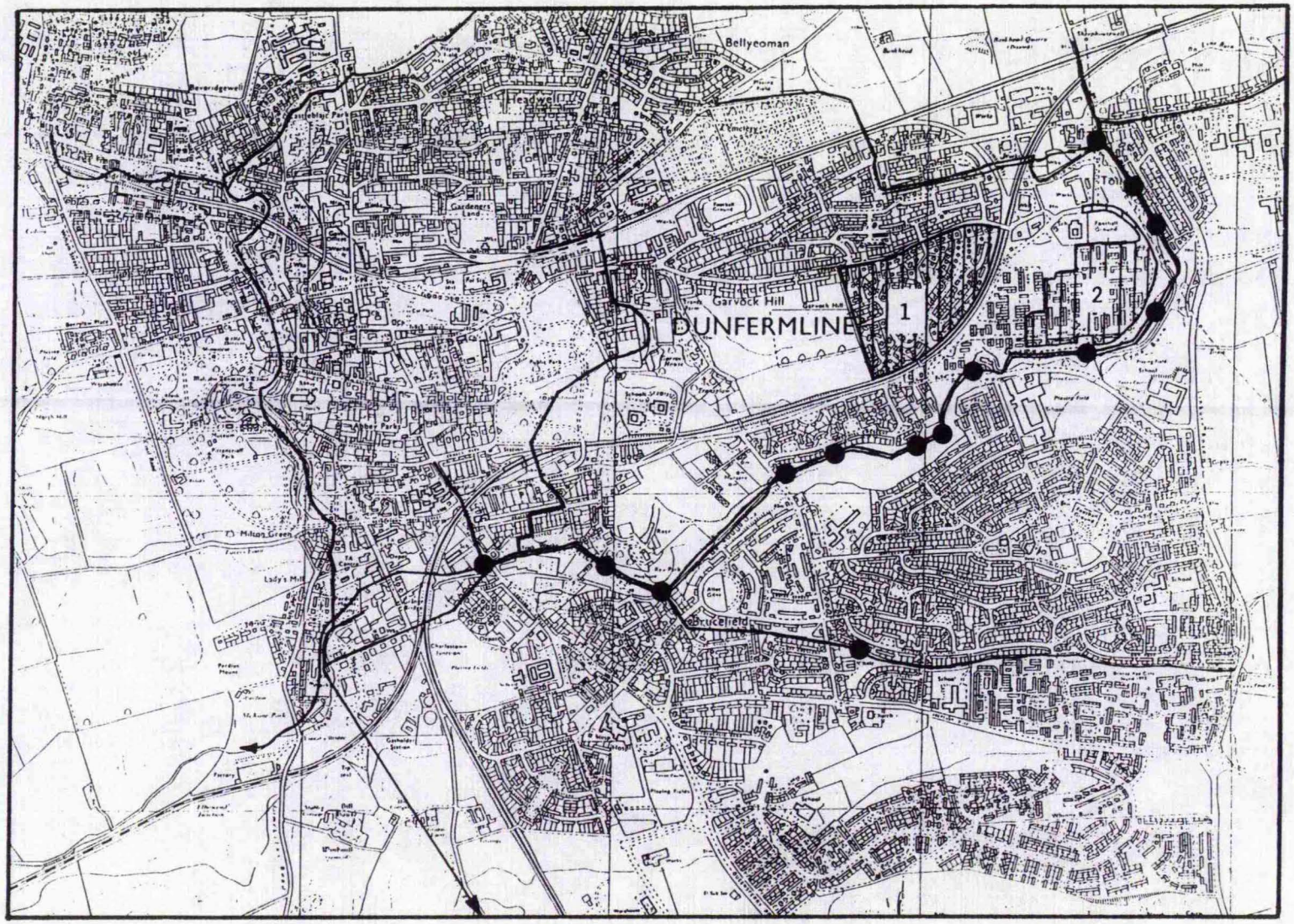


FIGURE 5.5 LOCATIONS OF SUB-AREAS FOR DETAILED OVERLAND SURVEY AND THE 'CRITICAL' SEWERS

Based upon the Ordinance Survey Map with the permission of the Controller of HMSO, Crown copyright reserved.

1 : Scotland Drive 2 : Garvock Bank • : Critical Sewers

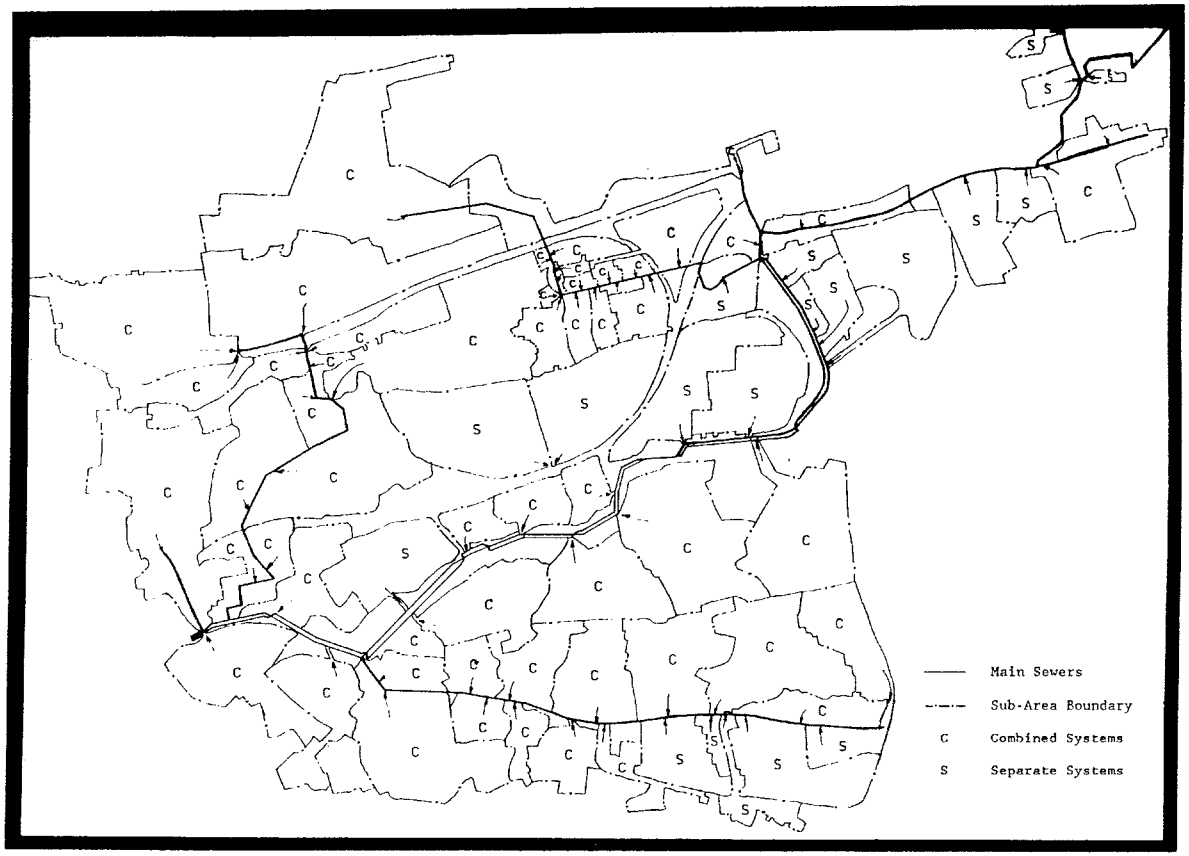
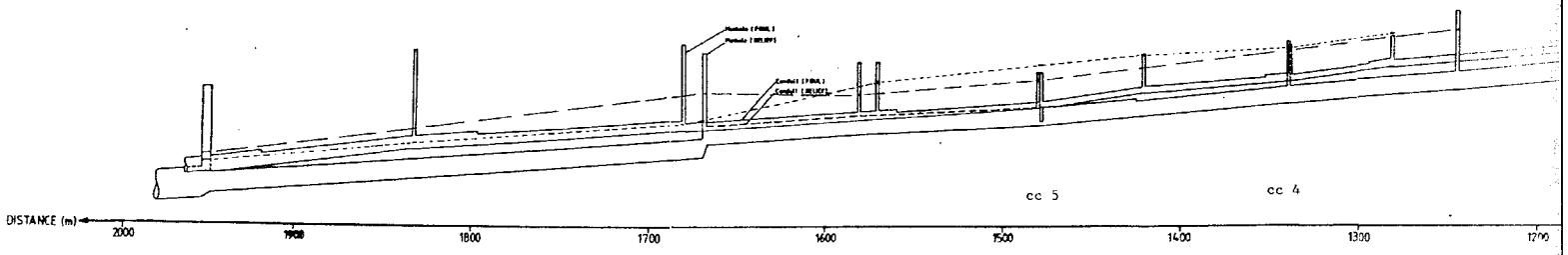


FIGURE 5.6 SEWERED SUB-AREAS



		Bothwell Street	Mill Road	Rex Park
LEVEL (m) :	Foul	0.699	2.206	1.935
	Storm	2.023	/	3.359
DISCHARGE :	Foul	288	143	188
(l/s)	Storm	3257	/	1810

FIGURE 5.7 Recorded Water Levels at Time 16:26 due to Total Rainfall of 32.6 mm on 17 June 1986

----- Foul Water Level
— — — Storm Water Level

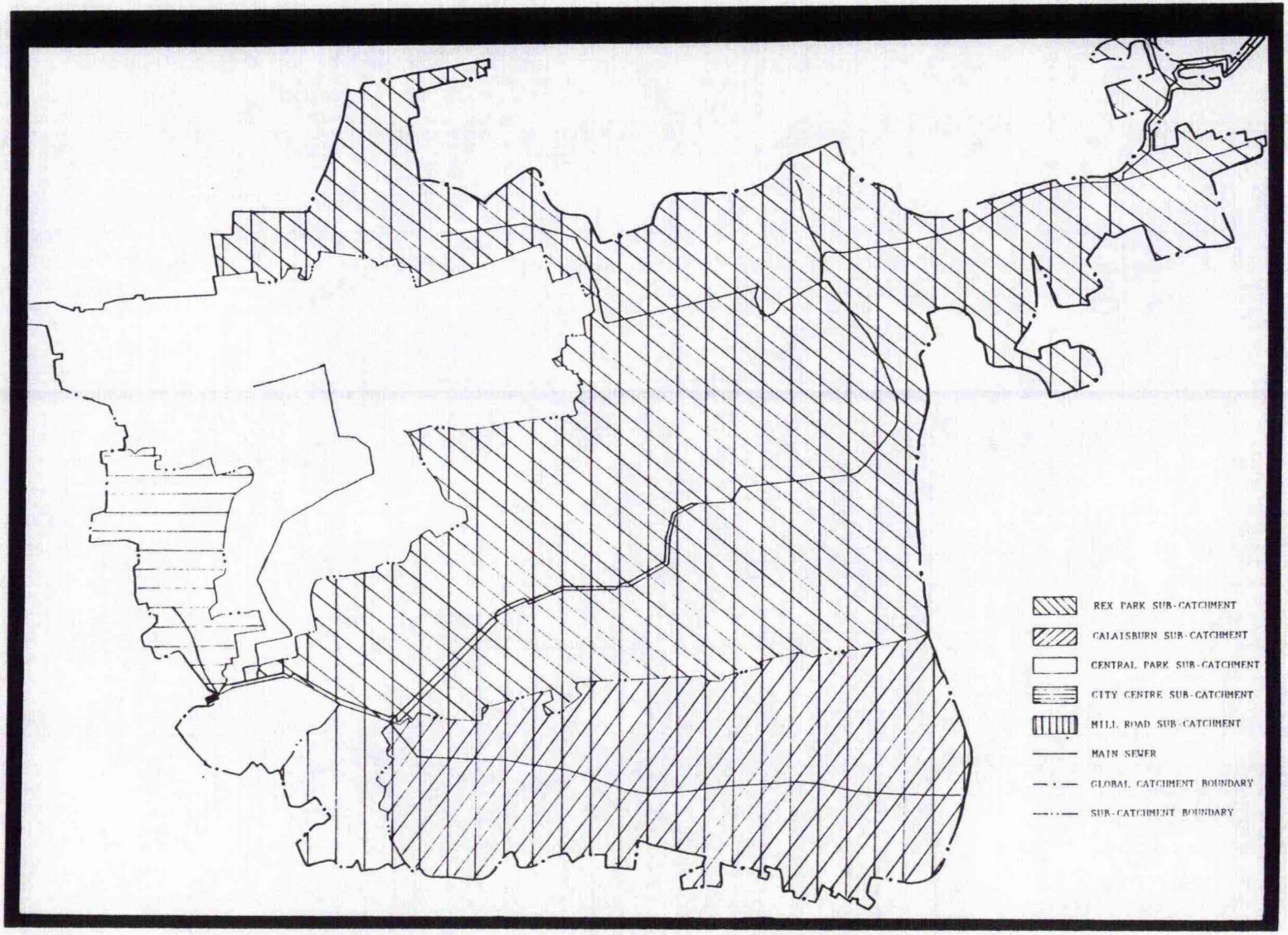


FIGURE 5.8 Global and Sub-Catchment Boundaries

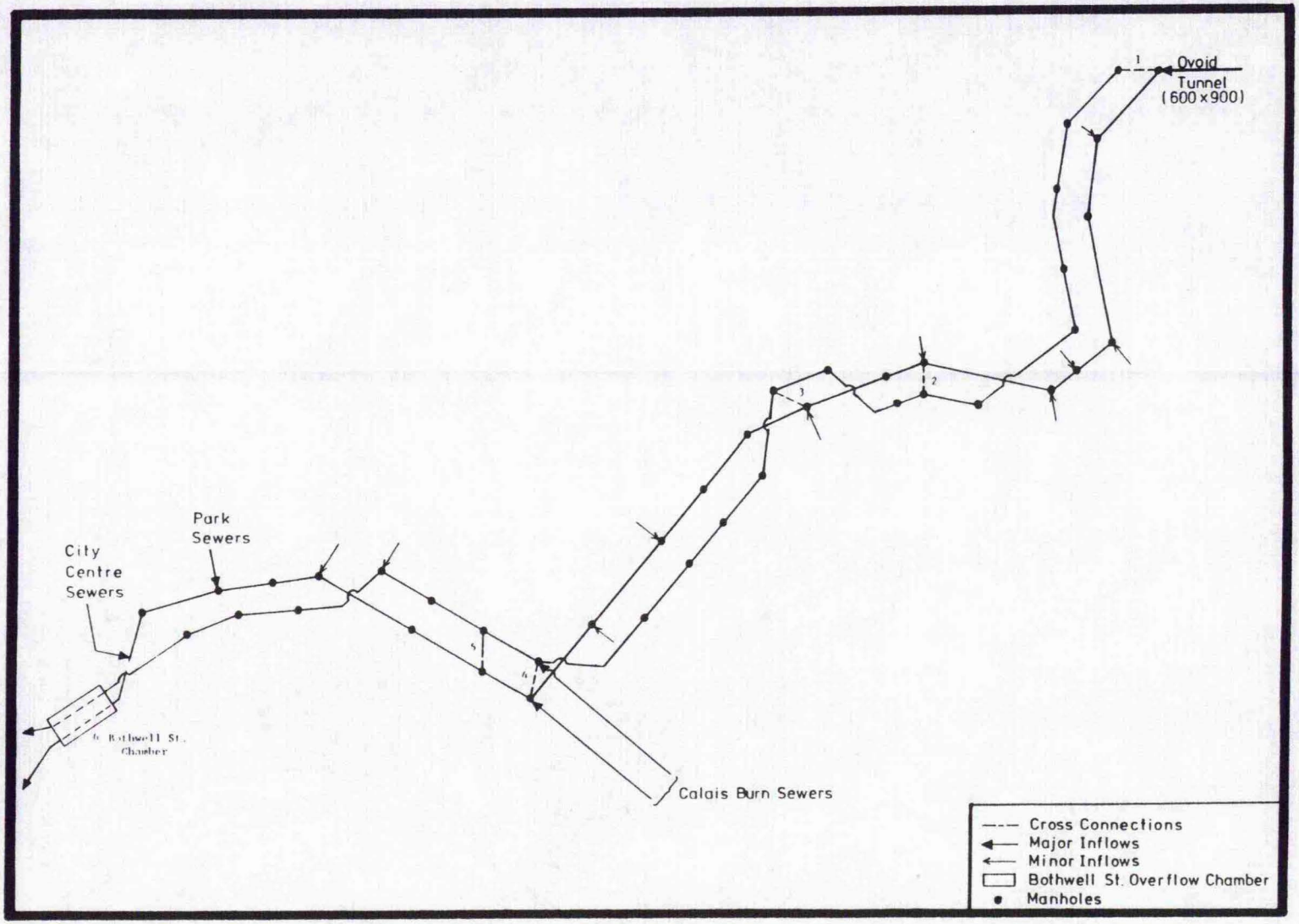
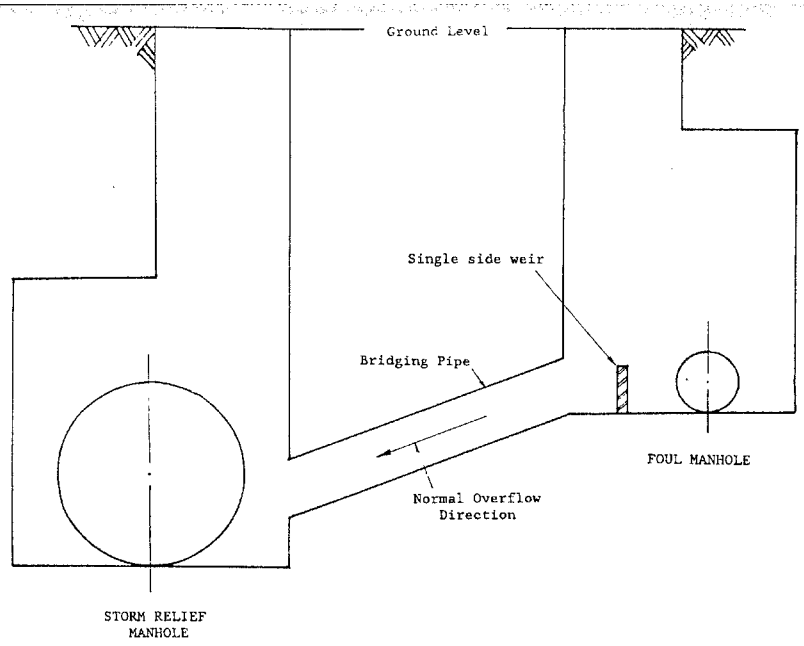
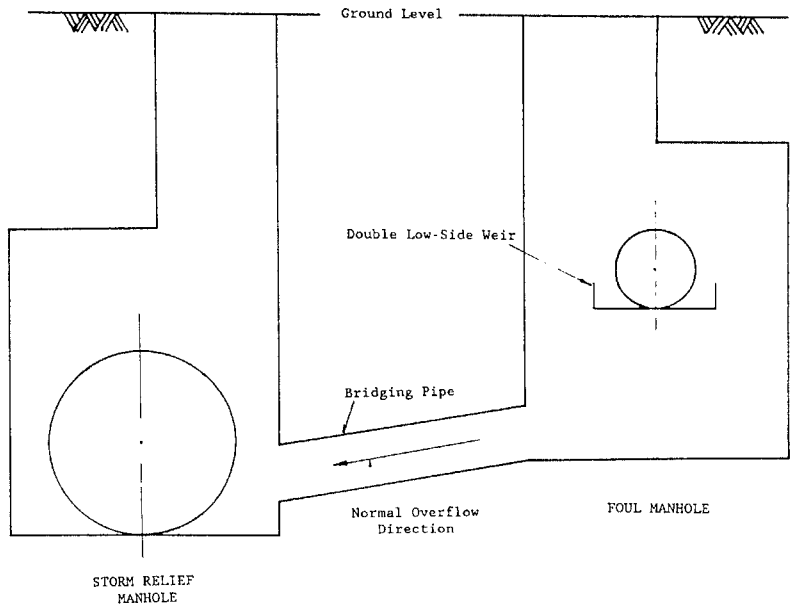


FIGURE 5.9 DETAIL LAYOUT OF PARALLEL SEWERS



(a) Cross-Connection Type I -- Single Side Weir Overflow



(b) Cross-Connection Type II -- Double Side Weirs Overflow

FIGURE 5.10 Diagrammatic Sketch of the Two Typical Types of Cross-Connection Overflows



PLATE 5.1 Dry Weather Flow in the Lyne Burn at Rex Park in March 1985

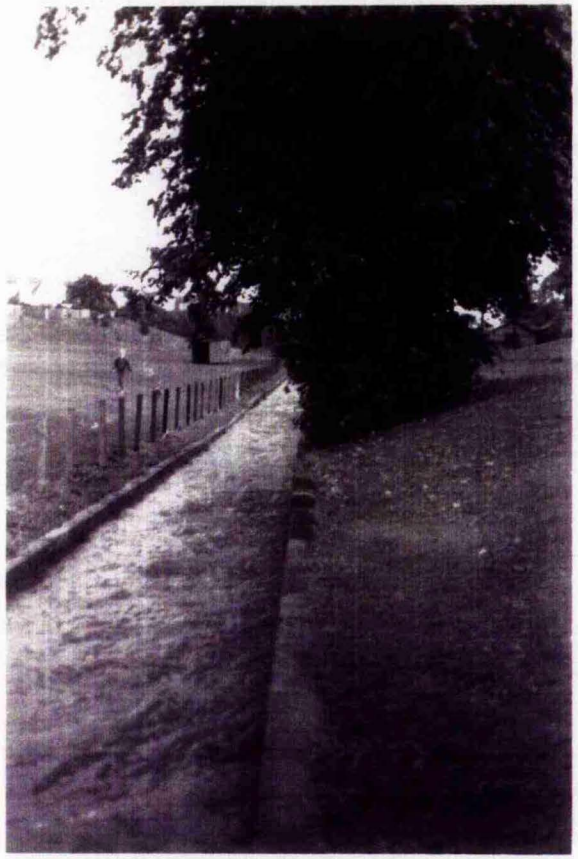


PLATE 5.2 High Flow in the Lyne Burn at Rex Park in May 1985



PLATE 5.3 Storm Outfall for a Housing Estate allows Combined Discharge into the Burn

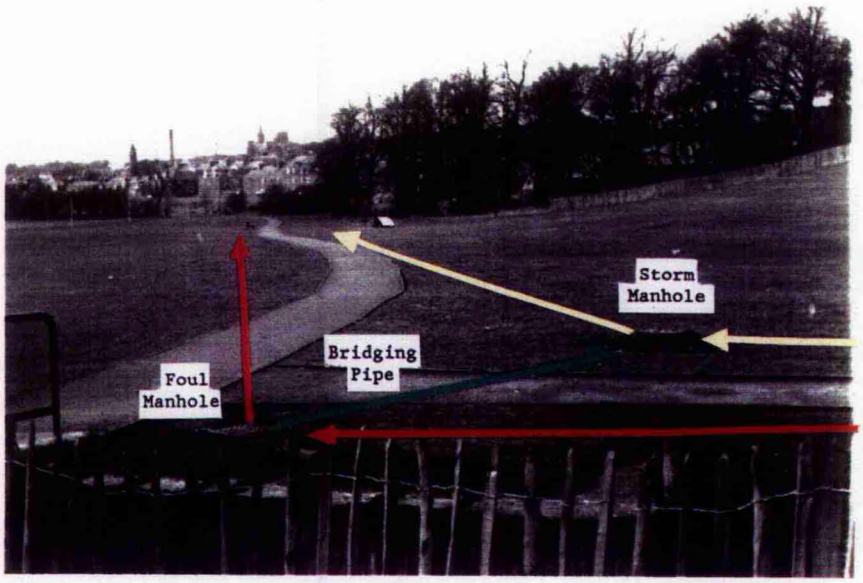


PLATE 5.4 Parallel Pipes and Cross-Connection No. 4 at Rex Park (see Figure 5.9)



PLATE 5.5 Surface Flooding Occurs Frequently at Rex Park due to the Shallow Manhole Depth and Mild Catchment Slope

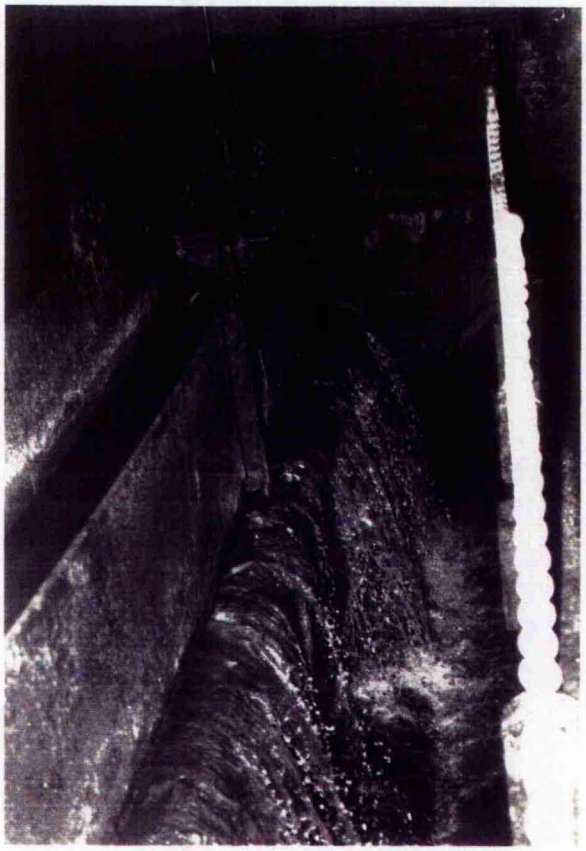


PLATE 5.6 An Overflow Occurred in Bothwell Street Chamber during a Storm on 1 April 1985

CHAPTER 6
INSTRUMENTATION AND DATA COLLECTION

6.1 INTRODUCTION

Modelling of rainfall-runoff processes on urban catchments has long been attempted in both theoretical and quantitative studies (Yen 1986). Early approaches took the runoff coefficient as equal to the proportion of impervious surface on a catchment by assuming that there was 100% runoff from impervious surfaces with zero from pervious surfaces (Watkins 1962). Many of the early empirical formulae estimated only peak discharge of storm runoff, the use of such relationship being mainly for the sizing of drainage pipes (HMSO 1963 & 1976).

In a fully-sewered urban catchment, the rainfall-runoff process can be conveniently divided into two areas: an above-ground phase comprising principally of hydrological phenomena and a below-ground phase primarily involving hydraulic pipe routing. In the past, simulations for both of these phases were difficult due to the lack of field data (Kidd 1978[2]). The appropriate step in the development of models had been to isolate each phase and to collect field data at the interface between the two (Sarginson 1973, Kidd 1976, Kidd & Helliwell 1977, Pratt & Henderson 1981). A comprehensive research effort involving investigation into overland flow and surface runoff phenomena has been performed and presented worldwide (Falk & Niemczynowicz 1978, Chevereau et al 1978, Helliwell et al 1976, Stoneham & Kidd 1977, Gunst & Kidd 1980, Maksimovic & Radojkovic 1986). Underground pipe flow behaviour has also been investigated extensively (Bettess & Price 1976, Thompson & Lupton 1978, Saul & Howarth 1982). Subsequently, many methods have been developed to provide additional information on rainfall-runoff processes such as the time distribution of runoff and also to facilitate a refined and more accurate representation of these processes (Colyer 1981, Pratt & Harrison 1982, Akan & Yen 1984).

In the UK, investigations into the hydrological processes was largely the responsibility of The Institute of Hydrology (Makin &

Kidd 1979, IOH 1980). Early instruments developed for urban drainage studies included the IH gully meter for measuring discharge through a road gully (Blyth & Kidd 1977), water-level sensors (WLS) for monitoring level in stilling basins (Strangeways & Templeman 1974) and water-level gauges for measuring level in conduits using floats and potentiometers (Verworn 1978). Nowadays depth and velocity are measured at the same time in sewers by means of ultrasonic sensors with computerised loggers (Green & Drinkwater 1985, Jacobsen 1983). Recently worldwide experimental catchments, where long-term rainfall-runoff is available, have been selected and summarised together with their simulation methods and models (Maksimovic & Radojkovic 1986).

The study of the Lyneburn drainage system involved the detailed measurements of both rainfall and in-sewer runoff. The equipment used for the data capture programmes is new in the field of urban drainage studies, such as the solid state computer loggers, and hence further investigations into this equipment is required.

6.2 MEASUREMENT OF RAINFALL

In the United Kingdom, rainfall records are received and recorded by the Meteorological Office from some 6,500 rain gauges scattered over Great Britain and Northern Ireland with the majority giving daily values of rainfall (Wilson 1983). In addition, there are a further 260 stations equipped with recording rain gauges which record continuously.

Standard daily rain gauges in Britain are made from copper and consist of a 5-inch diameter copper cylinder. The gauge has a chamfered upper edge which collects the rain allowing it to drain through a funnel into a removable container of metal or glass, from which the rain may be poured into a graduated glass measuring cylinder each day. The earlier recording or autographic rain gauges usually worked by having a clockwork-driven drum carrying a graph on which a pen records either the total weight of container

plus water collected, or a series of 'blips' made each time a small container of known capacity spills its contents. These gauges have the great advantage that they give intensity of rainfall directly.

Daily rainfall has been accurately monitored for over two hundred years, while recording gauges using tipping bucket and siphonic rain gauges have only been available for a number of years. Modern rainfall receivers are normally connected to a battery-powered electronic data logger which stores rainfall information prior to downloading to a field computer. These loggers use a quartz-based timer for recording the time of tips, and the data can be retrieved at intervals of one or two weeks.

In general, there are three types of recording medium now available and these are :

(i) ink recording on charts,
(ii) magnetic recording on cassettes,
(iii) direct recording into solid state chips.

Chart recording is now regarded as rather old-fashioned. The major advantage of this method is that a quick appreciation of the functioning of the instrument and of the events which have occurred is immediately available from a visual inspection of the record. However, the instrument depends on delicate mechanical parts in their recording mechanism. Furthermore, some effort normally is required to translate the chart record into digital form for subsequent use.

Cassettes and solid state recorders allow easy and rapid transfer of data from the recording site in the digital form and can be printed out for inspection. This recording technique requires computer installation for translation of the recorded data. This, therefore, often involves the effort to develop computer software to read and process the data. It is recommended that the stored data, which is stored in the solid state recorders, to be processed on-site in case of any malfunction in the equipment (Colyer 1982). Malfunctions inevitably involve the loss of data for several days or more. Frequent maintenance must also be carried out on-site and

this includes the checking of instruments. Solid state recorders usually have limited data storage capacity and relatively frequent visits are required to the site.

6.2.1 RAINGAUGES AND DATA LOGGERS

It is important that sufficient raingauges should be installed to enable an adequate coverage of a study catchment and to gain information concerning areal variability of rainfall. It has been shown that even on a catchment as small as 10ha, areal variation of rainfall can noticeably influence the runoff response due to certain events (Colyer 1982). WRc has suggested (Green & Drinkwater 1985) that a minimum of three raingauges should be installed within a catchment of 1100 ha. Two gauges are essential to provide an estimate of average rainfall whilst the third gauge is required to act as a backup. As a general guide, WRc suggests that an adequate raingauge coverage is one gauge per two square kilometres, subject to the minimum of three. In view of the size of Lyneburn sewered catchment (675 ha), and WRc recommendations (Figure 6.1), a minimum of four gauges were required for the purpose of model simulation.

For simulation purposes, a high resolution raingauge is always recommended (Makin & Kidd 1979). Two out of the four gauges used for the study were highly accurate Lambrecht 0.1mm gauges (Lambrecht 1982). The four gauges were :

(i) One 0.5mm tip IH daily totaliser gauge,
(ii) One 0.5mm tip gauge with paper chart recorder,
(iii) Two 0.1mm tip gauges with Squirrel loggers.

All the above are tipping-bucket raingauges. The two 0.5mm gauges acted as check gauges and all flow simulation relied on the Lambrecht 0.1mm tipping-bucket rain-gauges. The IH gauge was used to check the daily rainfall collected by the Lambrecht gauges. The 0.5mm tip gauge with chart recorder, however, was used to counter-check the captured events and also acted as a backup gauge.

The tipping bucket signals from the 0.1mm tip gauges were recorded by 'Squirrel' data loggers manufactured by the Grant Instrument Co. (Grant 1984). The loggers have a memory of 8K for data storage and recorded the number of tips in a constant time interval. Colyer (1982) suggested that choosing an appropriate time interval is important in relation to the bucket capacity. For a large bucket size the tipping frequency may be too low for accurate modelling. For 0.1mm bucket gauges, the suggested time interval range is from 4 seconds to about 3 minutes depending on the equipment used and the aims of the project. Due to the limited storage in the loggers, the time interval chosen was two minutes at the beginning of the study with data retrieved weekly. As confidence in the reliability of the instruments improved, time intervals of 4 minutes were chosen and data were retrieved fortnightly.

6.2.2 RAINGAUGE CALIBRATION

Both static and dynamic calibrations are required for each tipping bucket raingauge (Calder & Kidd 1978). Calibration is important because the amount of water held in each buckets may vary in response to changes in a number of variables. These are principally the height of the bucket stops, the resistance of the bearings and the time taken for the bucket to pass beneath the inlet stream of water.

There are two types of static calibrations, bucket balancing and number of tips. Balancing of buckets is carried out to check that the amount of water stored in each side of the bucket is identical before it tips (Parkar 1983). The calibration procedure is to drip water slowly down into the bucket until it just tilts whilst the amount of water is being noted. The same is repeated for the other bucket. The amount of water stored should be equal on each side and adjustment of the supporting screws may be required until a balance is achieved. Both the 0.1mm tip Lambrecht gauges used in the study stored approximately 2ml of water before the buckets tilted (Au-Yeung 1985[1]).

The aim of the number of tips calibration is to observe the number of tips over the specified time which should be the same as given in the user manual (Lambrecht 1982). Adjustment on the supporting screws may be required to achieve the correct number. To test the Lambrecht gauges, 200ml of water was released into the collecting funnel at a constant rate. The number of tips and total running time were recorded as 98 (gauge 1) and 96 (gauge 2) tips over a 15 minutes duration (Au-Yeung 1985[2]). The quoted figures were 96.7 tips over 15 minutes given in the manufacturer's manual. The results for both the gauges were considered to agree sufficiently well with the values shown in the manual.

High resolution and accuracy can only be achieved if raingauges are calibrated dynamically (Calder & Kidd 1978). As a first approximation, the static calibration may be assumed to relate the flow rate (Q) to the bucket capacity (V) and the time between successive tippings (T) as follows :

$$Q = V / T \qquad (6.1)$$

In general, a non-linear relation exists between flow rate and the tipping rate of a tipping bucket raingauge due to a variable quantity of water being lost during the time (t) taken for the bucket to move from rest to the central bucket position just under the inlet of water stream. If time t is assumed to remain constant, then the first-order dynamic calibration equation results as :

$$Q = V / (T - t) \qquad (6.2)$$

At low flow rates when time T is large compared with t, equation 6.2 is equivalent to the static equation 6.1. Hence the dynamic calibration only becomes significant at fast flowrates when tipping time T reduces.

A 'BBC' micro-computer equipped with an interface board was used to carry out the dynamic calibration. Variable flow rates were released from a Mariotte vessel which maintained a constant head throughout a particular test. The number of tips and the time taken for the bucket to tilt were recorded by the computer. Plate

6.1 shows the set-up for dynamic calibration. The outputs from the computer included the total number of tips, total running time, single tipping time and the corresponding flow rate or equivalent rainfall intensity. The test was repeated with a series of flow rates and a graph was plotted with rainfall intensities against the number of tips per 2 minute time interval for each gauge. Figure 6.2 shows the calibration results for the two 0.1mm gauges. It can be seen from Figure 6.2 that a linear relationship exists between the intensity and number of tips up to approximately 30 mm/h. The heaviest recorded intensity throughout the data capturing period for the research study was 28.0 mm/h which was still under the linear relationship and hence no adjustment to the observed intensities was required.

6.3 FLOW MONITORING EQUIPMENT

Measurement stations built into sewerage systems other than at inlets to treatment works are rare. To identify local or widespread problems within systems, it is therefore necessary to have a knowledge of the flow behaviour at the problem areas. The cost of putting permanent monitoring points in systems is prohibitive hence monitoring equipment must be portable. The advantage of having small and portable equipment is that it can be moved from location to location as necessary.

Flow monitoring in sewers has been attempted by drainage engineers for a long time. Up to the late 1970s this monitoring was usually undertaken manually by direct measurement using hand tapes at manholes, and dilution gauging was frequently used. The use of flumes and weirs in sewers was also common but this method had the severe drawback that calibration was lost once flumes were 'drowned'. Battery or clockwork 'dippers' were a later development providing the additional facility that flow levels could be continually monitored. In the early 1980s hand-held, battery-powered ultrasonic velocity meters became available commercially and it became practical to measure actual flow velocities directly

at several depths of flow. However, the use of handheld velocity probe is limited to calibration of the site's depth-discharge relationship.

Since 1976 the Water Research Centre (WRc) has been committed to assessing commercially available flow monitoring equipment (Williams 1984) and to encouraging manufacturers to produce suitable 'flow survey packages' (FSPs). The FSP instrumentation combines depth of flow and velocity monitoring into one housing unit and is considerably more reliable than the previous equipment. The housing unit or 'mouse' normally comprises a pressure transducer to measure depth and an ultrasonic velocity meter utilising the doppler shift principle to derive a reading for velocity (Ashley et al 1986, Au-Yeung 1989). These units have been readily accepted by the water industry and are now produced by several companies.

6.3.1 IN-SEWER FLOW LOGGERS

Many different types of flow survey loggers are available with varying degrees of sophistication. Specialist contractors offer complete survey contracts which include monitoring both sewer flow and rainfall. Some contractors have developed the data-handling aspects of their service to great sophistication or have acted as catalysts to this necessary development of equipment and data presentation. It is outside the aim of this study to describe all these methods, but useful discussions are found in Shelly and Kirkpatrick (1975) and Watts (1986).

To choose the most appropriate type of flow loggers for the Lyne-burn study, the following criteria had to be considered :

(i) The equipment had to be able to operate with the pipe flow both free surface and surcharged,
(ii) The mouse itself should not be too large that it affected the flow significantly by its installation,
(iii) The loggers should be portable for easy re-location,
(iv) The size of memory in the loggers should be large enough to store captured data for two weeks.

To meet the above requirements, flow loggers manufactured by Detectronic Ltd (1983) were chosen with two main components, logging unit and the sensor head or mouse. The logger units were dedicated micro-computers housed in uPVC tubular cansisters 430mm long by 230mm in diameter. The loggers were hung in manhole chambers with cables connecting from the base to the mouse located in the sewer just upstream of the chamber (Figure 6.3). The actual set up of the logger and mouse in the field are shown in Plates 6.2 and 6.3. It can be seen that the logger was hung just below the manhole cover in order to minimise the chance of damage due to surcharge (Plate 6.2).

The loggers have an internal memory capacity of 16K with recording time intervals ranging from 10 seconds to 30 minutes. User defined trigger levels allow a shorter recording time interval to be used once the depth is higher than a predefined level. This means that a 30 minutes interval, for example, can be used for the low or normal flow condition but a shorter interval, 2 minutes, is selected for events. The slow logging rate is resumed as soon as the flow depth falls back below the trigger level. This facility maximises the storage potential of the logger for monitoring flow during storms. The loggers are powered by two internal batteries with external rechargeable auxiliary batteries. The captured data is preserved and protected by an internal lithium cell in case of failure of the main power supply.

All the Detectronic flow survey loggers (shortened as DET loggers) used in the Lyneburn parallel pipe flow monitoring were set to 30 minutes for low flow logging rate and 2 minutes for the fast rate, with trigger levels being site-dependent. In addition to efficient use of data storage, this arrangement allowed the stored data to require interrogation only once every two weeks.

6.3.2 FLOW DETERMINATION

Depth of flow is measured using differential strain gauges which in turn measures the depth-dependent pressure exerted onto the transducer. Utilising the Doppler principle (Detectronic 1983) by a stream of pulses of very high frequency, sounds are emitted ahead

and the frequency shift of the echo is proportional to the flow velocity. Figure 6.4 shows a schematic sketch of the ultrasonic envelope for recording velocity : the region of measurement is a cone of angle of about 20^0 (Ashley & Jefferies 1988) to the horizontal line. If the depth profile changes rapidly the mean depth over the cone of velocity measurement will be substantially different from that recorded, leading to unquantifiable errors. A number of factors together with this lead to the necessary logger calibration covered in the next section.

Discharge is calculated using the following relationship :

$$Q = A \cdot Vav \qquad (6.3)$$

The cross-sectional area (A) is found using the recorded depth of flow. The following equation is used to determine the area in a circular pipe :

$$A = Ao \cdot \frac{\theta - [(1-2d / D)^2 \cdot \tan\theta]}{\pi} \qquad (6.4)$$

The above notations are shown in Figure 6.5. Equations 6.3 and 6.4 were enclosed in a computer program (Jefferies et al 1987) which was developed to compute from the observed data, that is depth and velocity of sewer flow, and the corresponding flowrates.

6.3.3 LOGGER CALIBRATION UNDER FREE SURFACE FLOW

There was little knowledge about the performance of the DET loggers at the beginning of the research study. Unlike the case for raingauges, standard procedures on the use of the flow survey loggers and their installation on-site were not available during that time apart from those in the user handbook. There was also no published material from independent users.

Only simple calibration tests were performed initially. Static depth checks were carried out to observe the difference between actual and recorded water levels. The results are shown in Figure 6.6. The percentage difference between the actual and recorded levels was found to be within the range ±2%, close to the

manufacturer's figure. Velocity tests were performed in a 300mm wide rectangular channel with variable inflow. The percentage difference between the actual and logged velocities was higher than the ±5% band, given in the user's manual. Furthermore, the difference in depth readings exceeded ±2% for those tests with greater velocities. The most extreme differences were observed when the depth was approximately 100mm or less (Au-Yeung 1985).

All loggers had been calibrated prior to their despatch by the manufacturer (Detectronic 1983). However, some of the DET loggers were initially found to be unreliable, failing to operate within a short time. As a result a programme was set up to check the loggers individually. A long series of tests to determine the limits of accuracy of the loggers were established (Au-Yeung & Ashley 1985). The aim of these tests was deliberately to look for errors and to set out the limits at which the equipment, in general, failed to measure the flowrate with sufficient accuracy. The test procedure was to separate the five loggers into two groups. Two were tested in a 300mm wide rectangular channel and three were tested with their sensor head in a 275mm diameter uPVC pipe flowing full. Plate 6.4 shows the pipe in the channel with its entry tapered to minimise tubulence, whilst Plate 6.5 shows the exit and weighing tank.

The flowrates determined by the laboratory equipment were then compared with those computed using the logged data. The percentage errors between the actual and the logged flowrates are plotted against the logged depth and the logged velocity (Figure 6.7). From these tests, it can be noted that percentage errors higher than ±20% were due either too low flow depth or low velocity, or both. The accuracy within ±20% could only be assured if the depth was greater than 100mm and the velocity was greater than 0.3m/s (Au-Yeung 1985). Both of these limits have been taken as guidance in choosing monitoring locations. These threshold values were confirmed with the aid of data provided by Southern Water Authority (Luck 1985, Ashley 1985). Some calibration tests for the DET loggers had been performed by WRc and similar results were obtained (Wilcox 1985).

6.3.4 LOGGER CALIBRATION UNDER SURCHARGED FLOW

After the free surface flow calibration described above, tests were carried out under surcharged conditions. A model culvert was built in the laboratory using a 300mm diameter clayware pipe in a large concrete flume in which full-bore velocities of over 1.5m/s with very stable velocity distributions could be produced (Butter 1986). With this arrangement a range of comparative tests on the velocity measurement was also carried out using hand-held turbine velocity meters. The results showed that the velocities given by the DET loggers and the hand-held ultrasonic meter were close to each other, but the propeller meter showed considerable differences. For most of the tests the difference between logged and laboratory recorded flow was within ±20% (Butter 1986).

6.3.5 GENERAL ACCURACY OF FLOW LOGGERS

The following observations can be drawn from both the free surface and surcharge flow calibrations :

(i) The loggers record depth at the sensor head by means of the pressure transducer but velocity is measured over a recording cone ahead, as indicated in Figure 6.4.

(ii) Depth measurement is generally accurate and reliable on its own. If the depth changes rapidly in the channel, the mean depth over the cone of velocity measurement will be substantially different from that recorded and hence large percentage errors in flowrate can result. Any steep logger installations should not to be considered in order to avoid rapid changes in depth along the channel.

(iii) Calibration results are better for those tests performed under surcharged flow condition. This is probably due to the lack of a free surface which can cause surface interference of the velocity measurement. The flow was stable under surcharged condition and velocity fluctuations were minimal.

(iv) Drift in measured depth will occur earlier than drift in velocity. This behaviour of loggers has been reported by other users and is acknowledged by the manufacturer.

The relative precision of these flow survey instruments has been considered by Wilcox (1985), Jefferies & Ashley (1985), and more recently by Ashley & Jefferies (1988) and Burrows et al (1989). The Flow Survey Manual published by WRc (1987) also set out the criteria for their use and recommended procedures to ensure that the transducers are functioning correctly.

In general these ultrasonic units are reliable and provide data which is within ±20% of the actual mean discharge at a given site with good hydraulic condition, that is without the presence of lateral inflows and free from back water flow (Wilcox 1985). On-site calibration can improve this figure to perhaps 10% but not normally over the whole range of flow conditions. The performance of these combined velocity and depth survey units still requires some deeper investigations over a boarder range of conditions in some typical sewers (Burrows et al 1989). More field monitored data from representative sites together with laboratory tests will be required in order to achieve this.

6.4 CATCHMENT DATA CAPTURE AND PROCESSING

The loading on sewerage systems derives principally from rainfall and waste-water flow. Other influences include infiltration, unknown connections, throttles and overflows, all of which make the actual distribution of flows within a sewer system different from that anticipated. Little progress can be made in any analysis without proper flow monitoring and sewer flow surveys thus have a major role for any investigation. Apart from the identification of possible unknown inflow sources, another important role of sewer flow monitoring is to verify the input data to models used for hydraulic analysis.

Storm and combined sewerage systems suffer their most severe loading during high intensity storms. By their very nature such

storms occur infrequently and they normally have considerable variation of intensity over the catchment and a number of rain gauges are required to cover an entire catchment. In the past the lack of suitable sites and gauges was one obstacle for sewerage monitoring. As only daily totals of rainfall were available, there was little attempt at monitoring sewage flows at short time intervals. In a fully developed urban area this poses particular problems since sites which meet the approved standards of the Meteorological Office are few (Baughen & Eadon 1983). Recently developed gauges, such as autographic type with tipping bucket and electronic signal receivers, provide the necessary rainfall data at shorter time intervals and information on variation of rainfall intensity.

Flow monitoring devices are also improving, leading towards the development of central data collection systems. Nowadays captured in-sewer flow records can be stored permanently on either magnetic tapes or computers. The development of databases using desk-top computers means that the captured information is easy to handle and process. The captured field data can be presented in both text and graphical formats and can even be readily accepted in computer models without further data processing.

6.4.1 EQUIPMENT LOCATIONS

Having determined the number and type of monitoring equipment available for rainfall and sewer flow measurement, the next step is to select locations for their installation within the catchment. Frequently this is based entirely upon practical site considerations.

A location for erecting a raingauge must be secure as well as exposed to the true rainfall pattern. A secure site is absolutely essential since experience has shown that raingauges are prone to vandalism (Green & Drinkwater 1985). However, secure locations are usually also sheltered and hence a practical compromise is required.

The 0.5mm gauge with chart recorder had already been installed before the research study and hence only three new sites were required for the raingauges mentioned in Section 6.2. A simple survey was carried out on plans to exclude the areas such as council housing, schools and public parks. Three locations were then selected followed by field visit. All four raingauge locations are shown in Figure 6.8 and Table 6.1(a) gives their names and site descriptions. The distance between the two 0.1mm gauges is approximately 1.2 km. Raingauge site A was chosen in order to support the DET logger at site 2 for the sub-area simulation (Chapter 7) and site B was located near the downstream end of the main parallel sewers. Photographs of the sites for the 0.1mm gauges and the 0.5mm totaliser gauges are included as Plates 6.6, 6.7 and 6.8 respectively. Only the locations of the 0.1mm gauges were considered to be completely satisfactory, the flat roof of the police headquarters being slightly over exposed and not at ground level.

Unlike the case for raingauges, the choice of sewer flow monitoring sites always causes less debate due to their below-ground installation in manholes. The procedure on selecting suitable sites is normally a two-stage process. Firstly, some general locations are selected according to the objectives of study, final selection normally being a compromise dependent upon local hydraulic conditions. A fuller description of the selection procedure for locations for sewer monitors can be found in the Sewer Flow Survey Manual (Green & Drinkwater 1985).

The below-ground sewer surveys mentioned in Chapter 5 uncovered some uncertainties and problem areas along the main parallel sewers. The final locations for the DET loggers were determined using the following general guidelines :

(i) At the parallel pipe system outfalls :
To measure the total outflow from the catchment. The percentage of flows in the twin pipes can also be established. Furthermore, a rapid check on the overall accuracy of simulation could be made and the significance of component inaccuracies could be assessed.

(ii) To avoid the problem locations :
Loggers should be installed only in those sewers free from hydraulic problems such as backwater and infiltration. Locations subject to rapid and sudden changes in flow depth are also not to be considered due to potentially high percentage errors in flowrate. Junction manholes with major inflows entering the main sewer are best avoided since the high flow from the major inflow tends to cause secondary velocities and sometimes reverse flow may also occur.

(iii) In a sewer which drains a small sub-catchment :
This is to give the flow response of a sub-catchment used for the construction of a simplified model based on the Sewered Sub-Area model. Logger installed at this location would only be a short term.

(iv) Upstream and downstream of overflows and cross-connections :
It is important to install loggers both upstream and downstream of cross-connections simultaneously so that the amount of overflow and changes in percentage of flows in the twin pipes can be determined.

(v) At locations along the main parallel sewer :
Various locations along the trunk parallel pipes were chosen to identify the proportional outflows at the different locations in the twin pipes. The amount of additional inflows from major sub-catchments can also be assessed.

(vi) At the outfalls of major sub-catchments :
To investigate the flow responses of the four major sub-catchments. These data were also used to check on the overall accuracy of the simulation outputs for each individual sub-catchment model.

A total of 14 logger locations were used around the system. One location, immediately downstream of the ovoid tunnel in which the parallel pipes merge (site 3), suffered from velocities over 2m/s and the flow depth was never greater than 120mm for the duration of its installation. As a result, the data could not be considered as being acceptably accurate according to the calibration results. Furthermore, 10 out of the 14 monitoring sites were on the separate

pipes of the parallel pipe systems. These could only be considered as 5 distinct locations when lumped pipe models were built. Consequently, 8 separate locations exist where significant lengths of flow records have been recorded. The 8 locations and their site numbers are listed in Table 6.1(b) together with a brief description for each site.

6.4.2 DURATION OF INSTALLATIONS

The number of loggers in use was small and this resulted in the flow survey being of the 'sequential' type with short period data (2-3 usable events) collected for representive sub-areas. Flows in the parallel pipes at the Bothwell Street chamber were monitored over virtually the full length of the study period. The guidelines in the flow survey manual (WRc 1987) suggest that a minimum of 16 loggers should have been used for a system the size of Dunfermline and that sequential flow surveys should only be used for 'minor' schemes. However even with the recommended 'fixed time and event dependent' type of survey, one particular event will rarely cover the entire catchment and it is felt that the approach to data collection used was justified and produced a satisfactory dataset. The verified models for the sub-areas presented in Chapter 7 show very good agreement between observed and predicted flows, vindicating the principle of the 'sequential' data collection method for the Lyneburn system.

Using the sequential logging method, a logger installed at a site would be re-located depending on the importance of the logging site and the number of events captured. In general, the duration of the logging period for a location was classified in three categories :

(i) LONG TERM
Continuous monitoring in which loggers stayed for the full duration of the data capture period in order to calibrate satisfactorily both the global catchment model and the parallel pipe model for which a substantial amount of data was required. Sites 1 and 7 fell in this category.

(ii) MEDIUM TERM

Continuous monitoring but for a period of several months in order to monitor the flow response of a sub-catchment. Sites 4 and 5 were chosen to be this type in order to facilitate the calibration of the area upstream of the parallel sewers without complication of major lateral inflows of the Calaisburn, Park and City Centre Sewers. The other location in this category was at Mill Road i.e. sites 8 and 9.

(iii) SHORT TERM

Site 2 belonged to this category and involved continuous monitoring of sewer flows over a period of only eight to twelve weeks, or two to three usable events in order to allow the Sewered Sub-Area model (SSA) to be calibrated. The other locations for short term flow surveys were the subsidiary inflows to the major catchment i.e. Calaisburn (sites 6 & 10), Park (site 12), City Centre (site 11) and Millhill Sewers (sites 13 & 14).

The lengths of time that the loggers were installed at the locations in Table 6.1(b) are shown in Table 6.2 together with the function of each sewer.

6.4.3 DATA MANAGEMENT

Rainfall is measured by the tipping-bucket raingauges. A pulse is sent via the reed switch to the Squirrel data logger whenever the bucket tilts. The captured data were stored in the logger's 8K internal memory and retrieved periodically using a portable Epson HX-20 microcomputer. The Epson computer has a built-in micro-printer and micro-cassette recorder from which the captured data could be printed out or stored on the micro-cassettes. In order not to lose data, the whole process including interrogation, data storage on micro-cassette should be completed before the next raingauge location is visited (Green & Drinkwater 1985).

Unfortunately, the pulse count loggers used up internal memory even when there was no rainfall, resulting in long sequences of zero

readings. The computer software supplied, SQUIRREL (Grant 1984), which was originally used only for data retrieval, had to be enhanced to filter out all but the usable rainfall data. The improved program could also print out the date, time and number of tips during an event. Figure 6.9 shows typical printouts using program SQUIRREL. The rainfall intensities were hand-calculated and for simulation purposes had to be typed in via the keyboard.

The DET flow survey loggers are in effect dedicated computers operated from batteries. Their operation is controlled using a keypad housed on top of the logger from where it can be programmed, started, stopped, reset and data interrogated.

The stored data were also retrieved using the Epson micro- computer via the RS-232C connection but with separate software, namely the program FLOW. This rather limited program would display the logged data in graphical format on both the LCD screen of the computer or on the micro-printer. Thus a quick visual inspection of the integrity of the data was made. Typical data from FLOW is shown in Figure 6.10 for both the levels and velocities at sites 1 and 7. These data were transfered to an Apricot Xi computer for processing.

The captured field data were transferred to the Apricot computer in separate stages using different software. Figure 6.11 is the flow chart showing the separate stages of data handling and equipment used, together with the computer software. As shown from the flow chart, the data set were then further transferred to the VAX mainframe computer for model simulations. Comparisons between the simulated and observed values in graphical forms were also possible in the VAX computer using a graph plotting program (Appendix B).

Data were stored in four different formats :

(i) Hardcopies on paper,
(ii) Micro-cassettes,
(iii) 3.5 inch floppy diskettes,
(iv) Magnetic tapes.

A central data handling and process computer suite, namely HYDROMASTER (Jefferies et al 1987), was later developed in the Institute. This interactive menu-driven package, consisting of all the software shown in Figure 6.11, not only speeded up the data transfer process but also gave consistency check on the captured flow data. Some of the rainfall and flow data for the Lyneburn catchment were processed by HYDROMASTER at the final stage of the data capture programme.

6.4.4. DATA ACCURACY AND CONSISTENCY CHECKS

In any field investigation it is important to carry out instrument checks frequently to ensure the captured data are accurate, meaningful and reliable. On-site checks were performed on both the raingauge loggers and DET flow survey units. In general the following were checked during each data retrieval visit :

(i) The general serviceability of the Squirrel loggers and DET flow monitoring units.
(ii) Any building up of silt, rags around the sensor heads and hence changes in hydraulic conditions for a particular sewer location since the last visit.
(iii) The reliability of the depth readings and the accuracy when compared with an actual measurement.
(iv) The reliability of the velocity readings and the accuracy when compared with a portable velocity meter.
(v) The correct operation of the data recording facility of all the electronic loggers.

A full set of site check sheets were prepared during each visit to Dunfermline to record the operating conditions of the logging equipment together with any additional information. A full set of the check sheets used for both rainfall and flow loggers are shown in Appendix F. Spot checks were carried out on site using hand-held velocity meter and a long wooden ruler. Rags and siltation around the sensor head were cleaned out regularly since the cables and the sensor head were twice found to be pulled apart, it was assumed because of the drag forces caused by rag collection.

The consistency of the recorded rainfall data monitored by the 0.1mm gauges was checked by comparison with the 0.5mm gauges. The 2 minute intensities monitored by the 0.1mm gauges were counter-checked against each other if possible. When only one Squirrel logger was operating, its data would be checked with that recorded by the 0.5mm chart recorder gauge. All the data recorded by the Squirrel loggers were found to be close, and less than 0.5mm different from daily total rainfall for most events. Close agreement between the two 0.1mm gauges was found. Table 6.3 gives a comparison of rainfalls for those events where both 0.1mm units operated. This table shows that both gauges recorded almost the same volume of rainfall despite being some 1.2km apart.

One of the facilities in HYDROMASTER was the flow data consistency check by producing scattergraphs. A set of data was plotted as a cluster of points on the graph and an envelope was drawn along the edges. These envelopes, representing a period of captured data, should be superimposed with each other indicating that the data were recorded in the same hydraulic conditions and the logger was operating satisfactorily. A databank was hence formed for each monitoring location. Any mislocation of the points or envelopes more than once would require to have the logger removed from the site for close inspection and recalibration. Figure 6.12 shows a typical scattergraph for site 1.

As a conclusion, both the rainfall and corresponding flow in sewers at various locations were monitored successfully throughout the data capturing period. Despite the distance of 1.2km apart between the two raingauge sites, areal rainfall variation was found to be small and similar rainfall depths were recorded. Sequential flow logging method was found to be an effective procedure in capturing flow data, especially when the number of logging units and available duration for monitoring were both limited.

6.4.5 AVAILABLE CAPTURED EVENTS

The monitoring of sewer flow started in early 1985 at site 3 (see Figure 6.8). The monitoring programme halted due to the breakdown

of one of the loggers at the time and data was considered to be unreliable due to an unacceptable flow condition at this site, i.e. fast velocity and too low flow depth. The data capture programme was resumed after more monitoring equipment was received.

The total duration of data collection was a period of 12 months from September 1985 until October 1986. This capture programme was prolonged slightly due to the inevitable logger breakdowns and shortage of working loggers to monitor all major sub-catchments. The summer of 1986 was relatively wet and a total of 32 usable events with rainfall depths higher than 5mm were recorded. Those events with total rainfall depths greater than 10mm are shown in Table 6.4 together with the sites where sewer flow data are available.

6.5 INTERPRETATION OF RAINFALL & RUNOFF DATA

All simulation models require the input of both the above-ground and below-ground sewer system data. Furthermore, some other important parameters which represent the global catchment, such as percentage runoff, depression storage and dry weather flow are required to be input to the simulation model.

Many of the above can be identified by carrying out surveys on both above-ground and in-sewer, but still some are required to be identified from the monitored rainfall-runoff data. Most obviously the complete storm flow response in sewers and corresponding peak flowrates caused by peak rainfall intensities have to be recorded. However, in addition, the sewer roughness and dry weather flows can be found from flow records alone, whilst the runoff coefficient, depression storage and percentage runoff can be established using both the rainfall and flow data.

6.5.1 PIPE ROUGHNESS VALUE

The sewer environment is very damp but relatively warm, conditions

which assist bacterial activity in the sewage and the formation of slime on the pipes. The effect of sliming on the roughness of sewers has been known for a long time (Perkins & Gardiner 1982, 1985). Experiments were carried out in 1979 by the Hydraulic Research Station to investigate the effects of slime growth on different pipe materials and a summary report was prepared (HRS 1979). The tests indicated that growth of slime in sewage happened within a matter of weeks. Furthermore, the pipe roughness (Ks) changes rapidly due to sliming over a wide range of different pipe materials. The report concluded that the greater the initial roughness of the pipe, the greater the slimed roughness. The roughness values for slimed sewers quoted in the latest edition of the HRS Tables (HRS 1983[1]) and Charts (HRS 1983[2]) are based on the results of these experiments.

A small pipe roughness value of 1.5mm was used initially, based on the report above, for the Lyneburn catchment simulation models but this resulted in excessive peak flows and runoff volumes. Graphs were then plotted with the recorded depths against velocities for two locations based on the normal flow condition (Figures 6.13, 6.14 and 6.15) and these were compared with those quoted in the HRS Charts and Tables (HRS 1983[1 2]). The three graphs have been plotted for the data from sites 2, 4 and 5.

Figure 6.13 for site 2 indicates that the points are located principally between Ks values of 3.0 and 6.0mm. Figures 6.14 and 6.15 for sites 5 and 4 respectively show that a Ks of 3.0mm is close to the observed data for the foul pipe (site 5) but those in the storm relief (site 4) are near to the 6.0mm.

In the later stages of model construction, the pipe roughness (Ks) values were based on those results obtained above. It was concluded that a roughness value of 6.0mm should be used for the lumped pipe model. For the enhanced parallel pipe model, Ks value of 3.0 and 6.0mm for the foul and storm relief pipes respectively should be used.

6.5.2 DRY WEATHER FLOWS

Apart from monitoring high flows due to the rainfall, the sewer flow records were also used to develop the normal dry weather flows for the sub-catchments and for various locations along the Lyneburn parallel pipes system.

Normally, dry weather flow for a catchment follows the daily, weekly and sometimes the seasonal variations in flow. This is particularly true for areas with extensive infiltration or exfiltration. In order to allow for the daily and weekly changes and to establish a better average value for dry weather flow at a particular monitoring location, sewer flow data for weekly periods without rainfall were plotted. A total of three different logging periods were plotted on the same graph to cover the seasonal variation.

Figures 6.16 to 6.18 show dry weather flow hydrographs for the locations along the parallel pipe system at Rex Park, Mill Road and Bothwell Street logging sites respectively. Hydrographs were plotted only for the foul pipe at Rex Park and Mill Road as there was no base flow in the storm relief pipe at these locations. In contrast there was a permanent baseflow in the storm relief pipe at Bothwell street and Figure 6.18 (a) and (b) show the flows in the foul and storm relief pipes respectively. It can be seen from these hydrographs that the base flow was lower in autumn (September and October) and higher in the spring (March). Furthermore, the dry weather flows in the foul pipe at Bothwell Street showed the least variation with very close maximum and minimum discharges daily for the three logging periods.

Dry weather flows for other locations have also been established by similar procedures. However, the logging periods were generally shorter for these locations. Table 6.5 summarises the dry weather flow values for all flow logging locations. The data also showed that the average base flow related to the catchment size was 0.1 l/s per hectare.

6.5.3 DEPRESSION STORAGE AND RUNOFF COEFFICIENT

Depression storage (DEPSTOG), the storage of rainwater on the surface of the catchment, and the runoff coefficient (RC), the constant proportional loss after initial losses, have been defined and investigated by previous researchers (Kidd 1978[1], Pratt & Henderson 1981). Both parameters, together with the runoff volume (RUNVOL) are highly significant in modelling overland runoff process. In the Wallingford Procedure WASSP (HRS 1981), depression storage is calculated for nine surface types and for sloping roofs. The net rainfall is calculated by subtracting the relevant value of depression storage at the start of the storm but adding the average storage value back on to the remainder of the storm in order to maintain the same rainfall volume (HRS 1981). The volume and distribution of runoff is then determined by a two-part runoff model. The first calculates the volume of runoff on the ground surface and its distrbution in time and space, whilst the second routes this runoff as it flows overland before entering the sewer system.

The overland flow models developed by the Institute of Hydrology (Stoneham & Kidd 1977, Kidd 1978[2]) for the Wallingford Procedure were based on a very limited data set which was not representative of all types of catchment (Pratt 1984). Regression equations have in reality little to do with engineering, for example the percentage runoff (PR) equation implies that roof runoff is affected by soil type (Williams 1985). The latest variant of WASSP is the WALLRUS (HRS 1988) software which incorporates some minor changes in the overland flow model such as the option for variable percentage runoff value for every contributing pipe area. The way in which the rate and volume of runoff are modelled remains virtually unchanged.

The long sequence of sewer flow monitoring in this study has enabled the relationship of depression storage and the runoff coefficient to be evaluated for each sub-catchment of the Lyneburn system. These values have been compared with various locations in Dundee (Au Yeung et al 1989) and other catchment data (Kidd 1978[1],

Pratt & Harrison 1982). RUNVOL is determined from actual data by producing a plot of rainfall (P) against observed in-sewer flow, regression analysis is then used for the following relationship (Pratt 1985) :

$$RUNVOL = RC\ (P - DEPSTOG) \qquad (6.5)$$

The graphs of rainfall against runoff volume are plotted for all sub-catchments and for the full catchment at Bothwell Street and are shown in Figures 6.19 (a to e). The RUNVOL relationship in the form of Equation 6.6 is shown on each graph. The values of DEPSTOG and RC are summarised in Table 6.6 for all locations. It can be seen from this table that the DEPSTOG values are very similar for those locations along the parallel pipes except the Calaisburn and Central Park sub-catchments. This is probably due to insufficient rainfall-runoff data. The RC values range from 0.24 for Calaisburn to 0.8 for Rex Park catchment. It is probable that the high RC for Rex Park is due to the higher density of housing estates and paved areas. Despite the dense housing in the Calaisburn catchment, there is a greater pervious area and some separate systems drain to the Burn at the upstream end of the system.

6.5.4 PERCENTAGE RUNOFF

The amount of runoff from a catchment is very much dependent on the depth and duration of the rainfall, the initial catchment wetness condition and the soil type. Percentage Runoff (PR) from rural areas in the UK in the past would usually be derived with the aid of equations in the Flood Studies Report (NERC 1975) in the form of :

$$PR = 95.5\ SOIL + 0.22\ (CWI-12) + 0.1\ (P-10) + 12\ URBAN \qquad (6.6)$$

This equation has been modified and is presented in Supplementary Reports (Folland et al 1981, IOH 1983). The improved equations are based on data from 130 catchments with most data derived from

winter storms. However, runoff from rural areas differs markedly from urban areas, in particular the below-ground flow (Packman 1981). The equation for PR derived for use in WASSP following analyses of urban catchments has the form as below :

$$PR = 0.829\ PIMP + 25.0\ SOIL + 0.078\ UCWI - 20.7 \qquad (6.7)$$

Equation 6.7 was developed based on data from 17 catchments, each less than 2.5 km^2 in area, and having 'impervious' areas of 20% to 70%. Most of the data collected for the development of equation 6.7 were from summer storms.

Simulation software such as WASSP and DUCTS allow a site determined value of the catchment percentage runoff (PR) to be input and to replace the catchment data and thus Equation 6.7 becomes :

$$PR = SPR - 0.078\ UCWI \qquad (6.8)$$

With sufficient rainfall-runoff data, an average standard percentage runoff (SPR) value for a catchment can be evaluated using the following regression equation for a catchment :

$$SPR = \frac{RUNVOL \cdot 100}{P \cdot AREAC} \qquad (6.8)$$

Equation 6.9 is in fact the same as Equation 6.5 with RC expressed in percentage. The PR values can hence be found from the graphs shown in Figures 6.19 (a-e) for all subsequent sub-catchments and again summarised in Table 6.6. All these percentage runoff values are used for both the lumped-pipe and parallel-pipe models for all catchments.

6.5.5 FLOWS AT CROSS-CONNECTIONS

Due to the limited number of DET flow monitor loggers, none of the five cross-connections were fully monitored. However, the data captured at various locations along the parallel pipes indicated that overflows occurred at the cross-connections for rainfall depths as small as 5mm. Reverse flow was not normal due to the

difference in the invert levels of foul and storm pipes in the upstream part of the parallel pipe system. However, head balance in both foul and storm manholes was possible at the downstream cross-connections (numbers 4 and 5, see Figure 5.9 for their locations) due to the shallow manhole depths (Plate 5.4 and 5.5).

Flows during the heavy event which occurred on 17 June 1986 with a rainfall depth of 32.6mm were recorded at Rex Park and Bothwell Street overflow chamber. The recorded flowrates and other details are shown in Figure 5.7. An interpretation of these recorded sewer flow data between these two sites indicated that a balance of the flow depths in both foul and storm manholes occurred as shown in Figure 6.20.

Although reverse flow had not been shown from the above data interpretation, it was actually possible, as flow in the storm relief manhole during events was much greater than in the foul, and hence a higher head in the storm side at cross-connections 4 and 5 could occur. The possible flow balance and reverse flow phenomena in the cross-connections forms a major component of the simulation of the parallel pipe system. The enhanced simulation model has also been developed to incorporate the different flow regimes in the cross-connections.

SITE REF.	SITE NAME	DESCRIPTION
A	Old Kirk Place	Local resident's back garden
B	Brucefield House	Local resident's front garden
C	Police Headquarter	Roof of a 3-storey Police Headquarter building
D	McKane's Park	Local resident's back garden

(a) Raingauge Sites

SITE NO.	LOCATION	DESCRIPTION
2	Scotland Drive	A small sub-catchment used for initial model testing.
4 & 5	Rex Park	Two loggers. Upstream from this point the catchment area is relatively homo-geneous
6 & 10	Calais Burn	Two loggers were located at Wallace Street for a short period on this major contributing sewer.
8 & 9	Mill Road	Two loggers on the Lyneburn system to determine the characteristics of the parallel system.
13 & 14	Mill Hill Street	A major contributing area.
12	Park Sewer	A second branch on the north side.
11	City Centre	A logger was located on one pipe from the city centre.
1 & 7	Bothwell Street	Two loggers were installed for most of the duration of the study on the inlets to the overflow chamber.

(b) Logger Locations

TABLE 6.1 Locations of Raingauge and Logger and Their Descriptions

No.	SITE NAME	SEWER TYPE	DURATION
1	Bothwell Street Chamber	Foul	30/9/85 -- 27/10/86
7		Relief	9/12/85 -- 27/10/86
2	Scotland Drive	Relief	30/9/85 -- 2/12/85
4	Rex Park	Relief	30/9/85 -- 31/7/86
5		Foul	30/9/85 -- 20/6/86
6	Wallace Street	Relief	31/7/86 -- 1/9/86
10		Foul	31/7/86 -- 1/9/86
8	Mill Road	Foul	13/6/86 -- 29/9/86
9		Relief	16/7/86 -- 29/9/86
11	Erskine/Beveridge Court	Relief	1/9/86 -- 29/9/86
12	Erskine/Beveridge Court	Combined	1/9/86 -- 27/10/86
13	Millhill Street	Relief	29/9/86 -- 27/10/86
14		Foul	29/9/86 -- 27/10/86

TABLE 6.2 Logger Locations and Duration of Monitoring

EVENT DATE	RAINFALL CAPTURED BY 0.1mm RAINGAUGE (mm)	
	OLD KIRK PLACE (Location A)	BRUCEFIELD HOUSE (Location B)
851109	9.2	9.6
860130	7.9	8.2
860323	9.5	9.1
860422	13.1	13.5
860510	14.5	14.4
860512	5.6	5.7
860806	8.9	9.1
861019	8.1	7.9
861019(B)	4.8	5.0
861027	5.6	5.4

TABLE 6.3 Comparison of Total Rainfall Captured at the Two Raingauge Sites

DATE	RAIN (mm)	SUB-CATCHMENT / CATCHEMENT					
		SCOT DRIVE	REX PARK	CALAIS BURN	MILL ROAD	PARK SEWER	BOTHWELL STREET
30/ 9/85	19.5	*	*				
9/11/85	10.9	*	*				
30/11/85	23.4	*	*				
15/ 4/86	26.5		*				*
20/ 4/86	12.9		*				*
22/ 4/86	13.5						*
7/ 5/86	16.2						*
10/ 5/86	14.5		*				*
17/ 5/86	11.1		*				*
10/ 6/86	16.8						*
17/ 6/86	32.6		*				*
28/ 7/86	14.4				*		*
16/ 8/86	11.3			*	*		*
2/ 9/86	18.1				*	*	*

TABLE 6.4 Catchments with the Available Events

LOCATION	BASEFLOW (l/s)	TOTAL CONTRIBUTING AREA (ha)	BASEFLOW (l/s per ha
Rex Park	36	326	0.110
Calais Burn	8	157	0.051
Mill Road	44	492	0.089
Central Park	20	177	0.113
Bothwell Street	71	675	0.105

TABLE 6.5 Dry Weather Flows at Flow Monitoring Locations

LOCATION	DEPRESSION STORAGE (DEPSTOG) (mm)	RUNOFF COEFFICIENT (RC)
Rex Park	2.84	0.80
Calais Burn	0.21	0.24
Mill Road	2.99	0.40
Central Park	0.92	0.63
Bothwell Street	3.36	0.62

TABLE 6.6 Values of DEPSTOG and RC for Flow Monitoring Locations

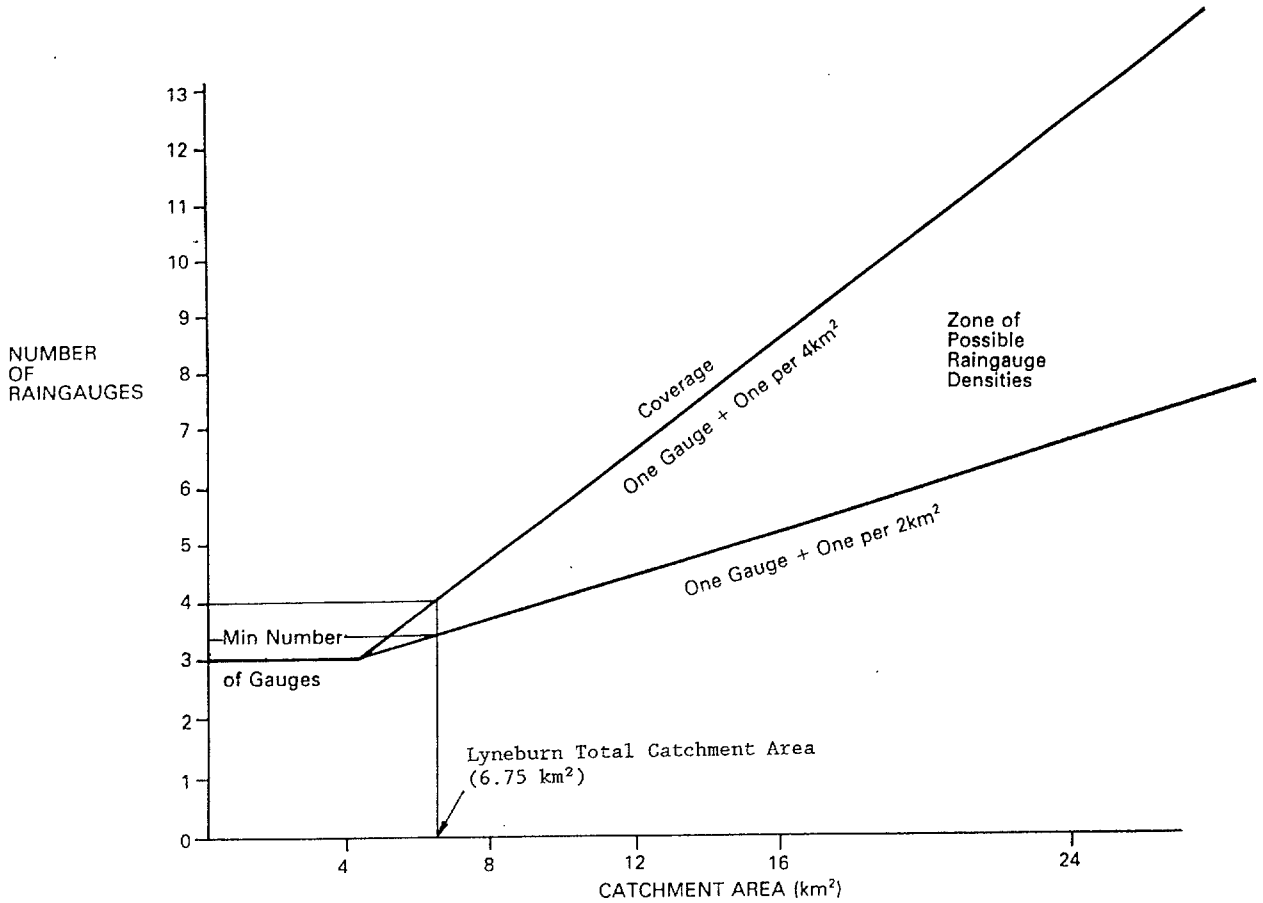


FIGURE 6.1 SUGGESTED NUMBERS OF RAINGAUGE FOR LYNEBURN CATCHMENT (WRc 1985)

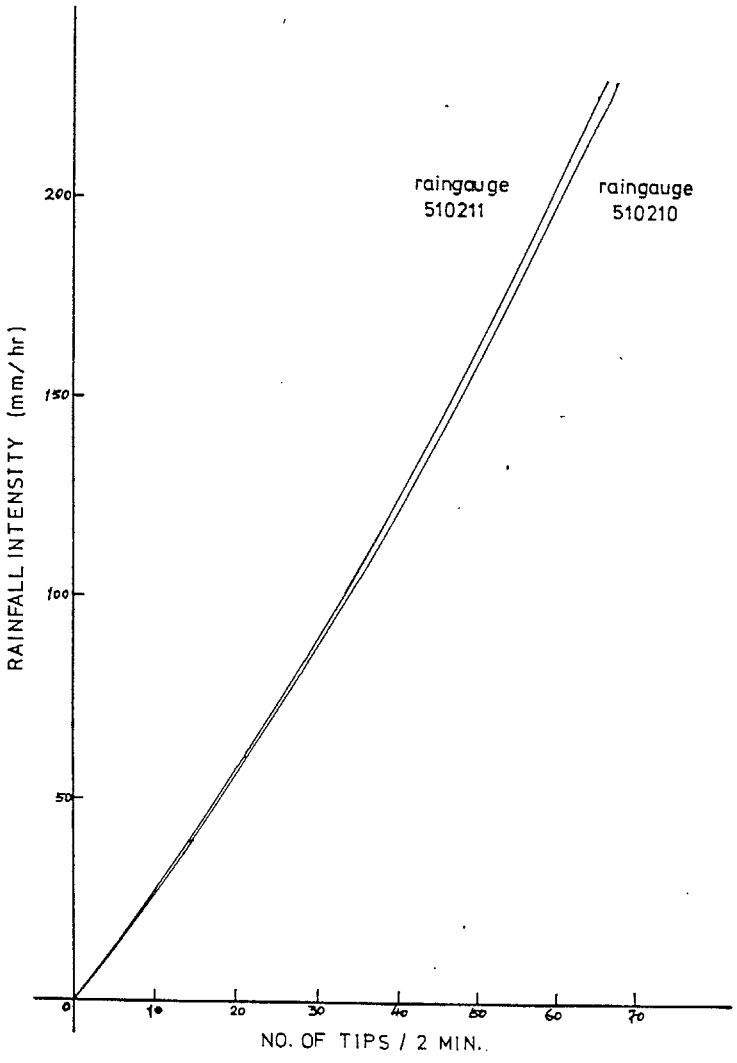


FIGURE 6.2 DYNAMIC CALIBRATION RESULTS FOR LAMBRECHT 0.1mm TIPPING-BUCKET RAINGAUGES

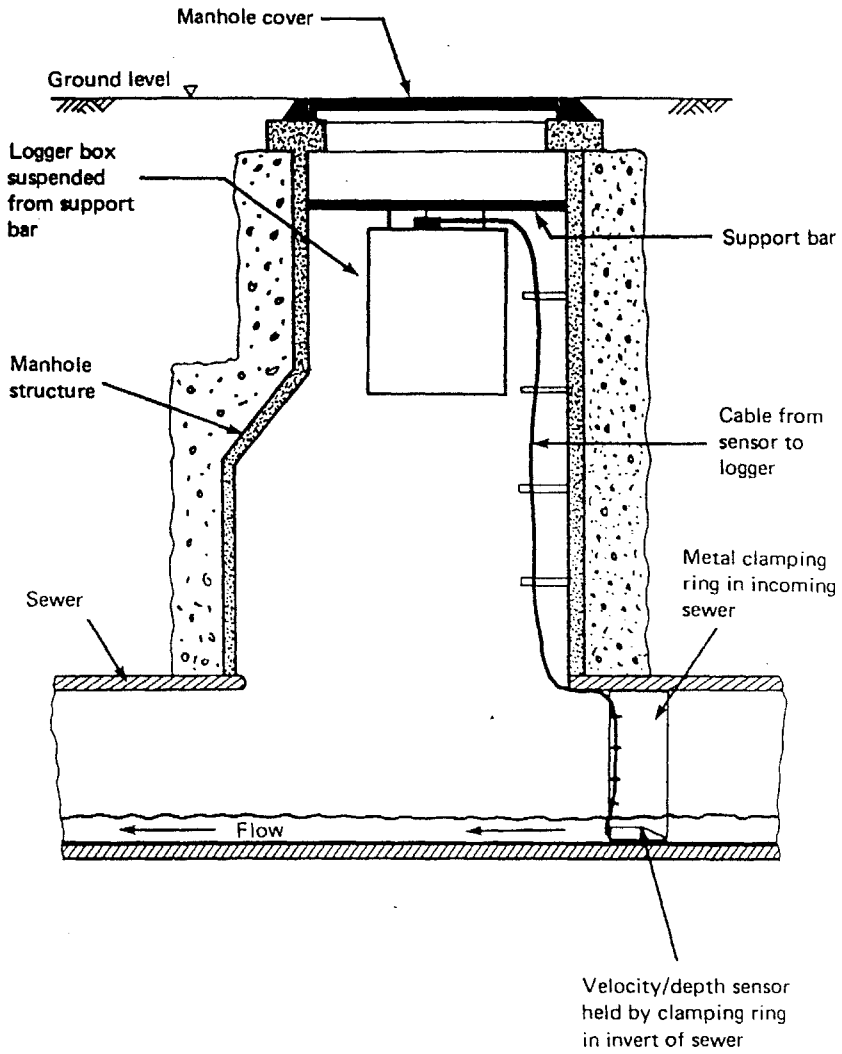


FIGURE 6.3 INSTALLATION OF SEWER FLOW SURVEY UNIT (WRc 1985)

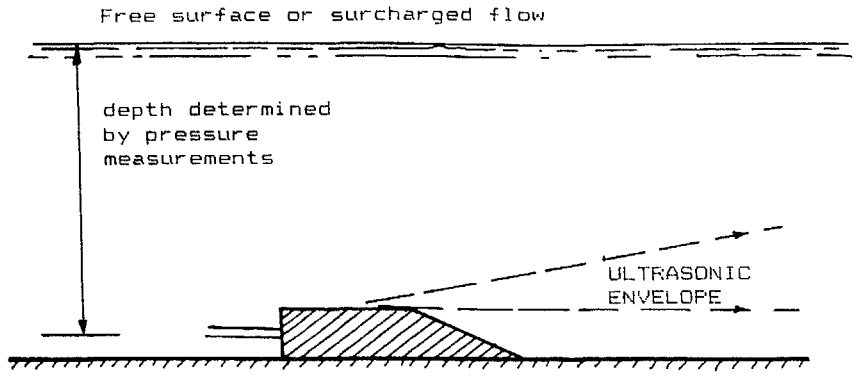


FIGURE 6.4 FLOW LOGGER SENSOR HEAD

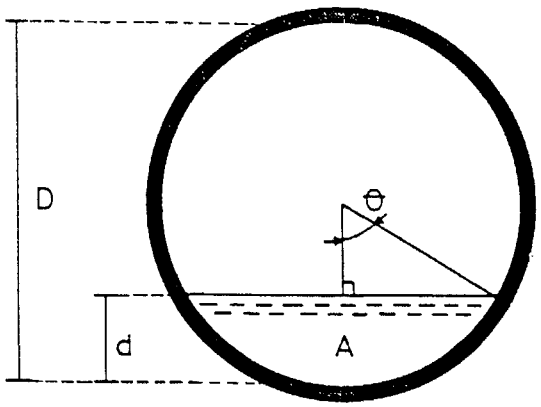


FIGURE 6.5 THE PARAMETERS OF THE CIRCULAR CROSS-SECTION

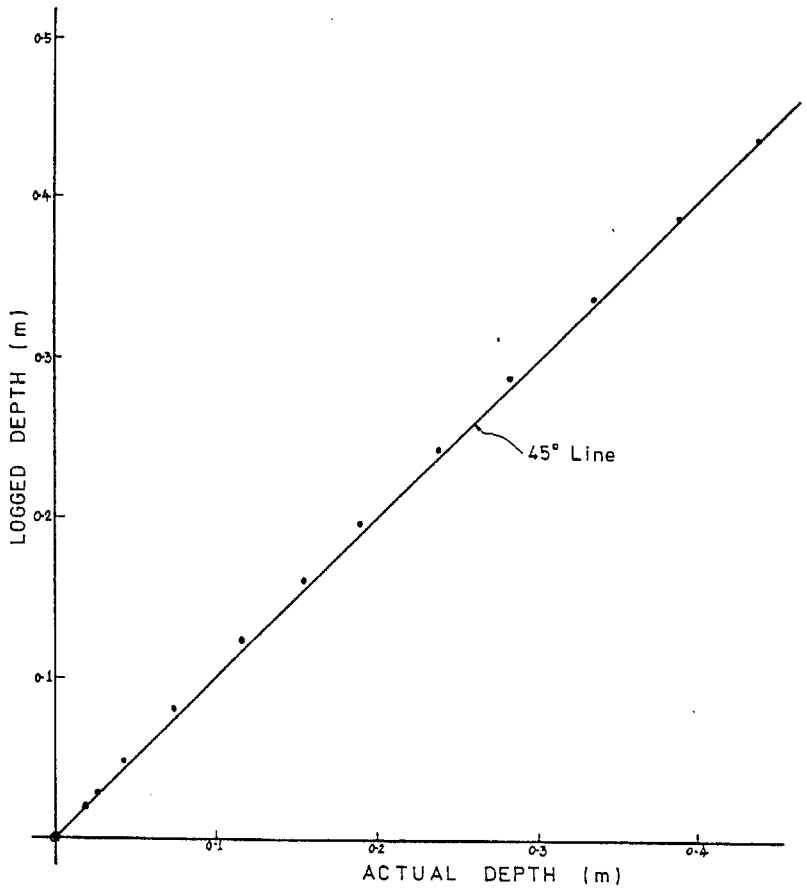


FIGURE 6.6 STATIC DEPTH CHECK FOR DET SEWER FLOW SURVEY LOGGER

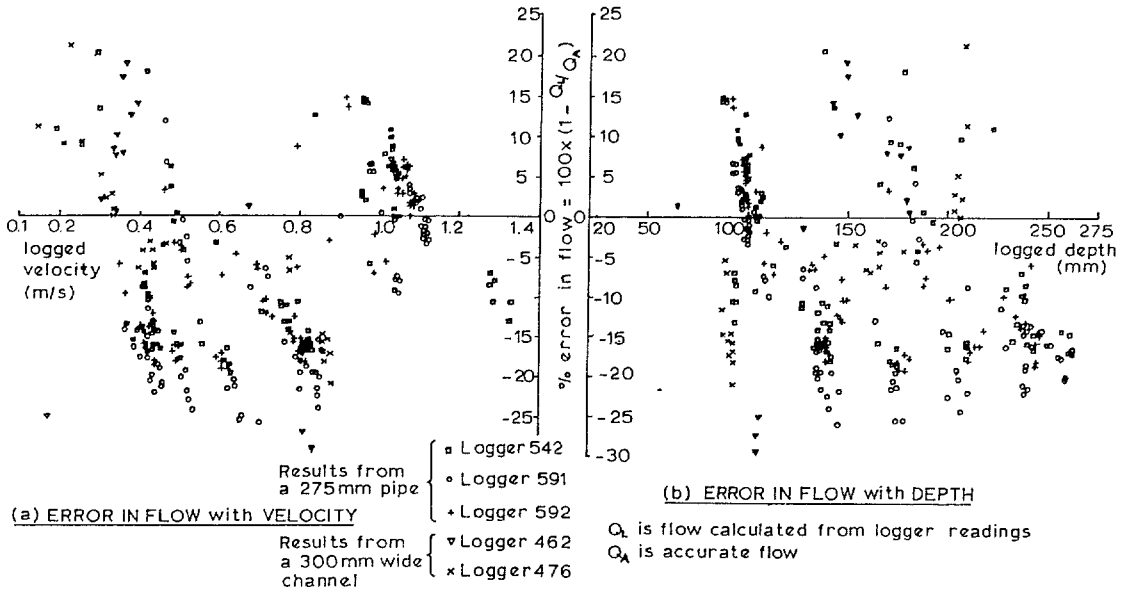
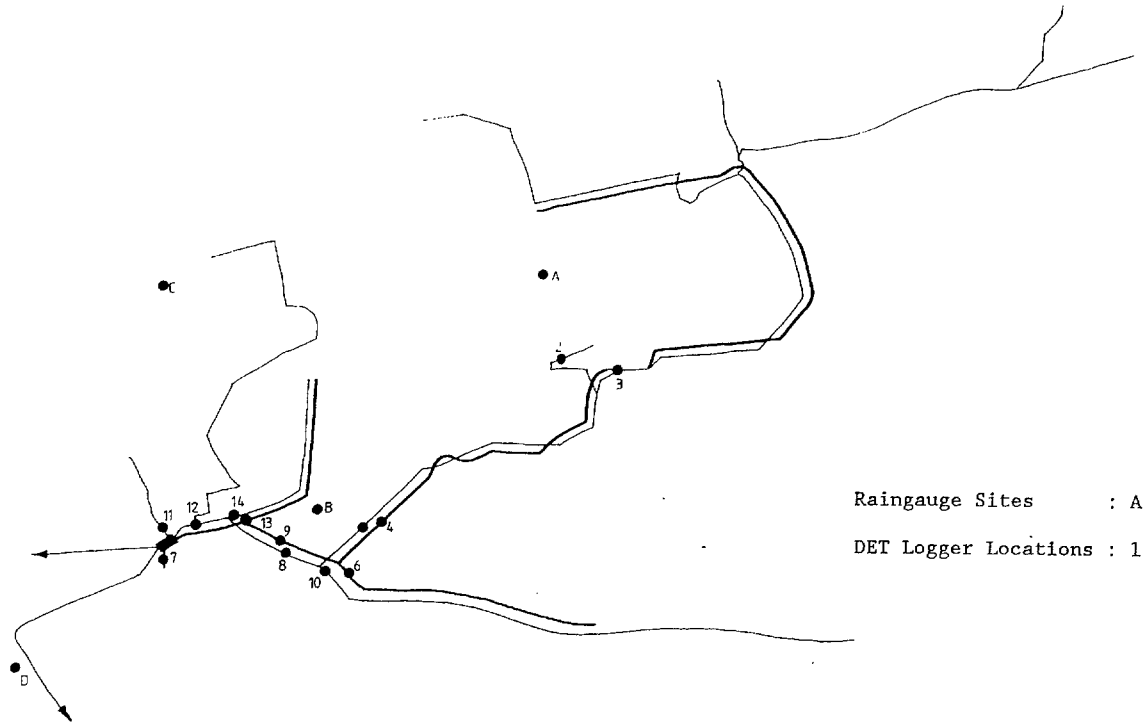


FIGURE 6.7 PERCENTAGE ERRORS IN DEPTH AND VELOCITY FOR DET FLOW SURVEY LOGGERS

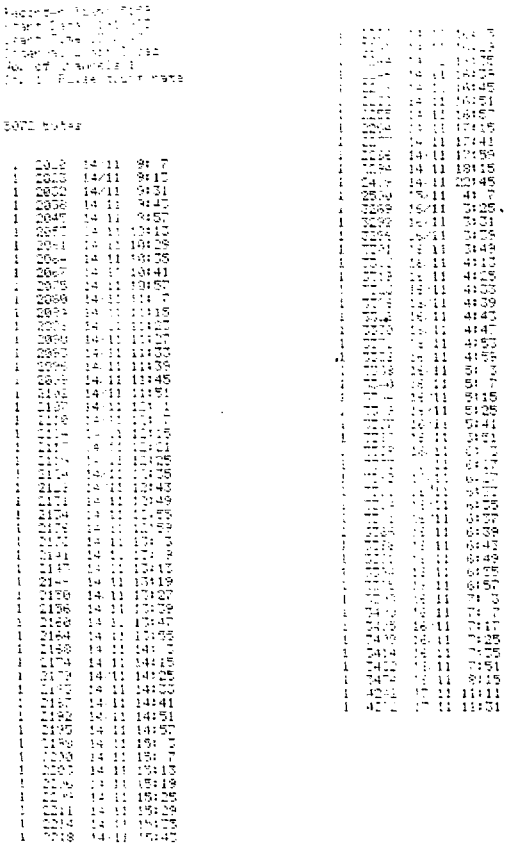


Raingauge Sites : A – D

DET Logger Locations : 1 – 14

FIGURE 6.8 LOCATIONS OF DATA CAPTURING EQUIPMENT

Rainfall data collected from Site B - Brucefield House



NOTES: 1st column is number of tips / 2 minute interval
2nd column is internal counter (for checksum only)
3rd column is the date
4th column is the time
Zero tippings are not shown in the printouts.

FIGURE 6.9 TYPICAL RAINFALL DATA PRINTOUTS

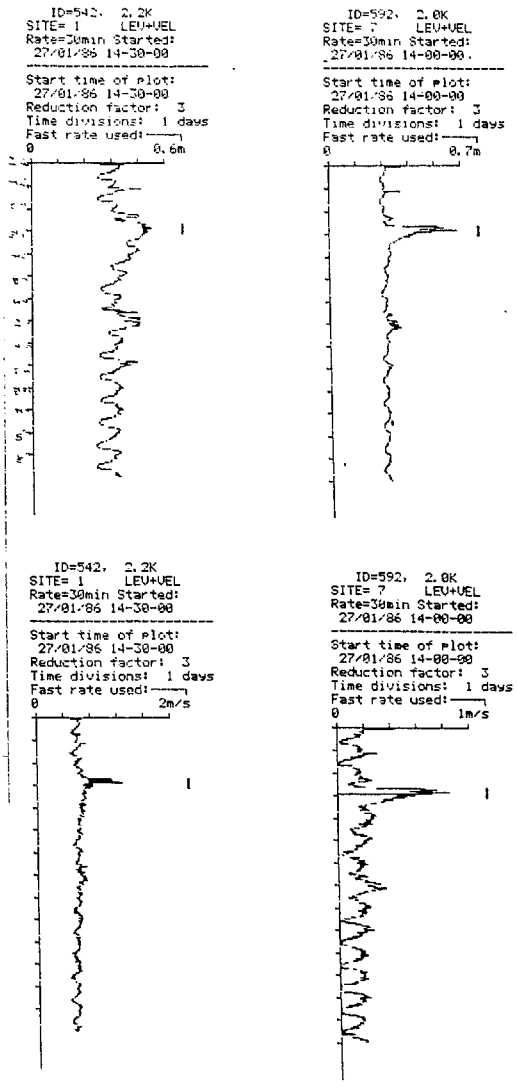


FIGURE 6.10 GRAPHICAL REPRESENTATION OF RECORDED SEWER FLOW DATA

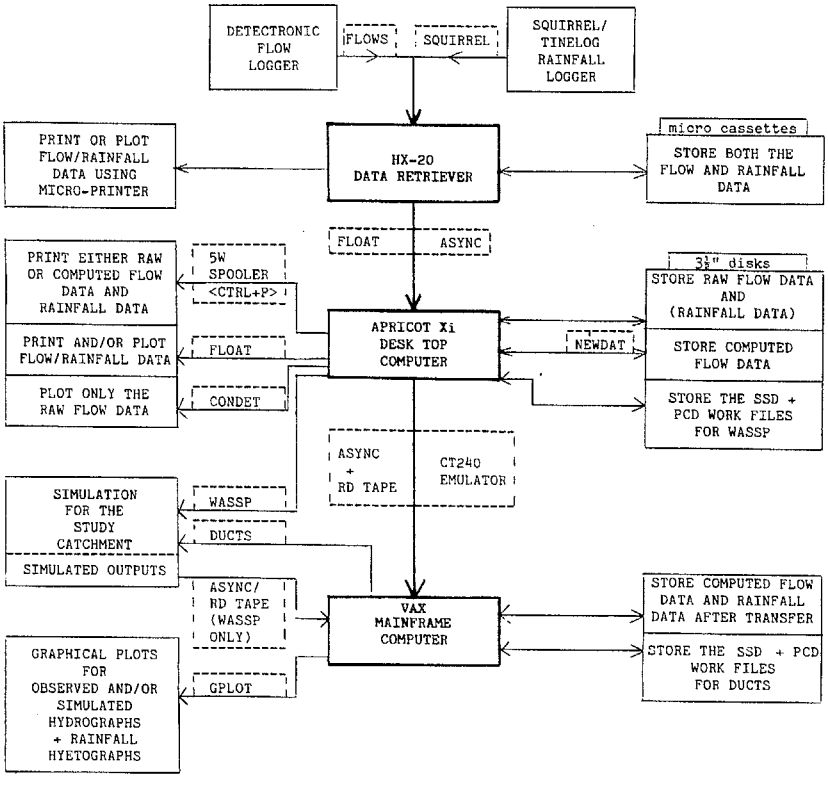


FIGURE 6.11 DATA HANDLING PROCEDURE AND THE ASSOCIATED COMPUTER SOFTWARE

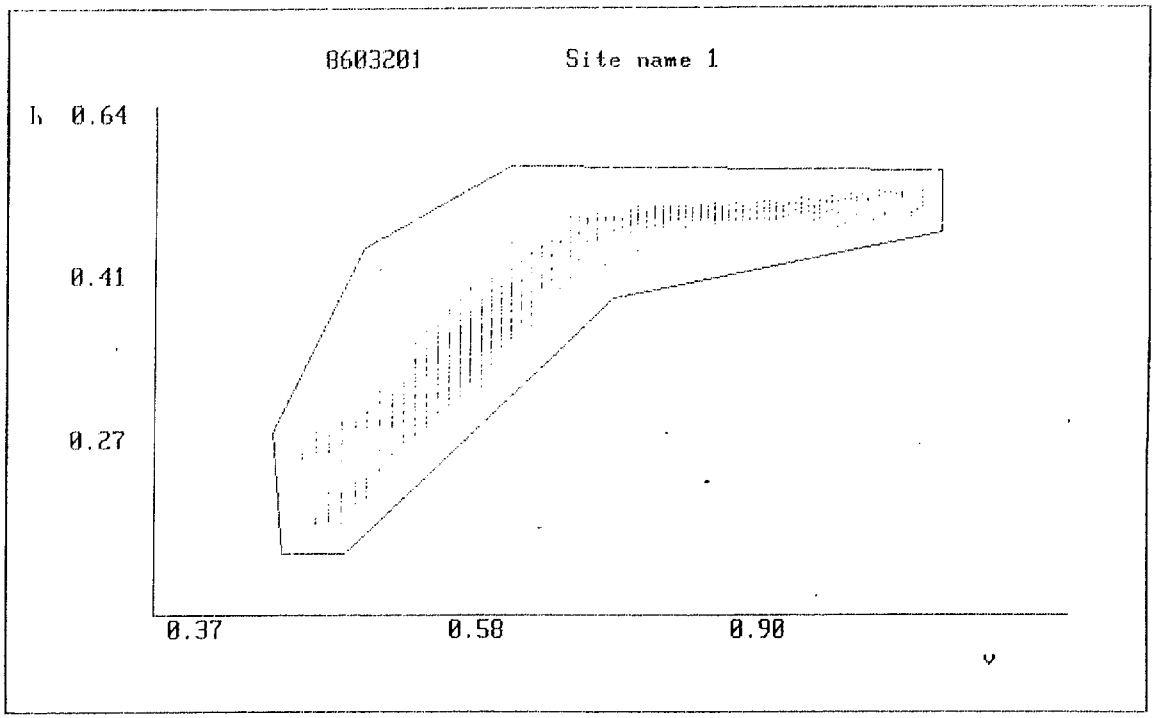


FIGURE 6.12 TYPICAL SCATTERGRAPH PLOT (Bothwell Street -- foul)

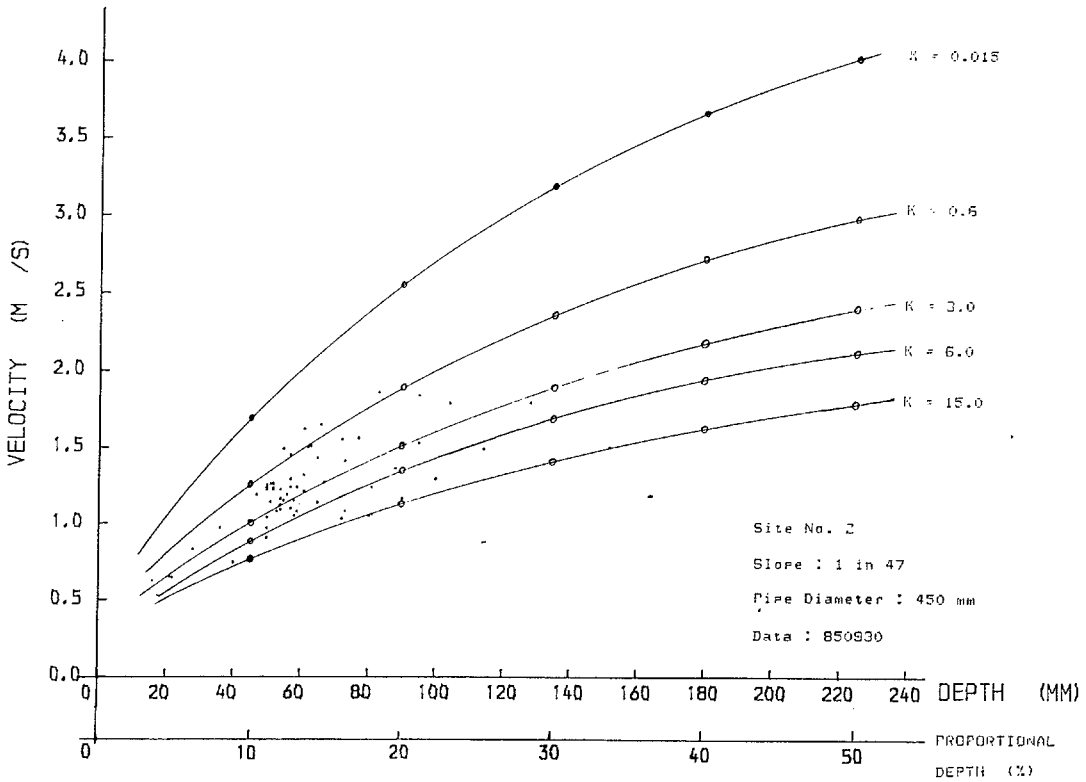


FIGURE 6.13 ROUGHNESS (Ks) DETERMINATION FOR SCOTLAND DRIVE

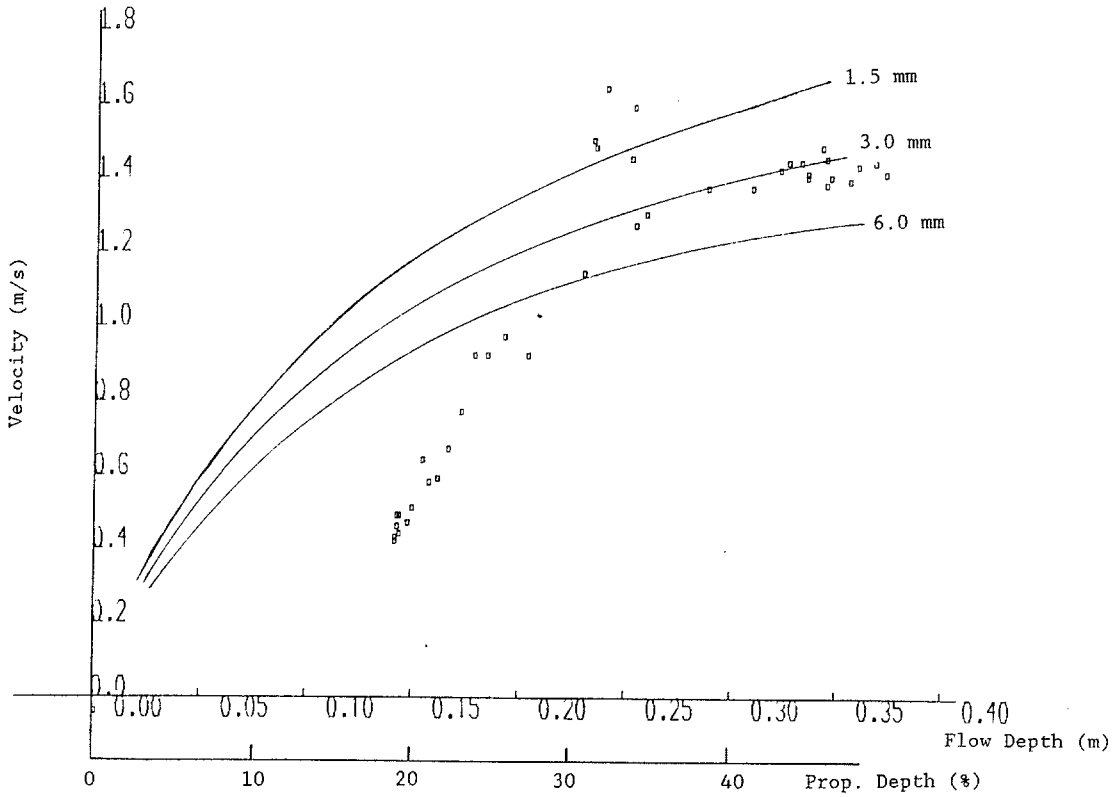


FIGURE 6.14 ROUGHNESS (Ks) DETERMINATION FOR REX PARK (Foul)

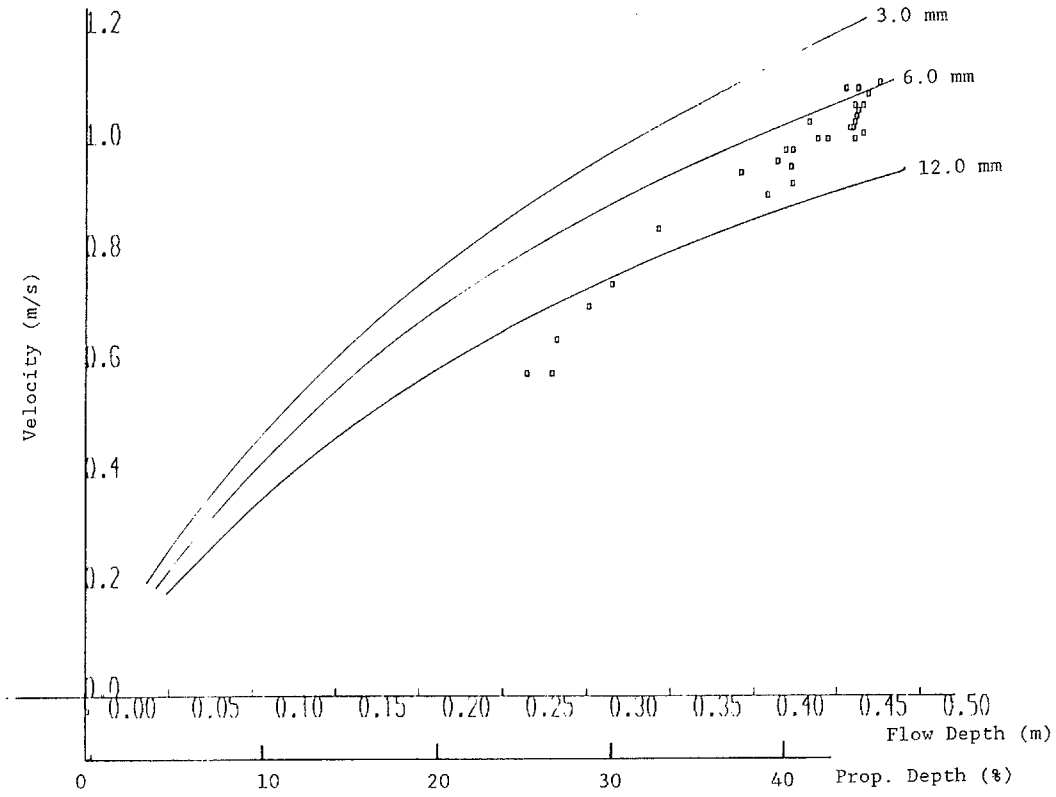


FIGURE 6.15 ROUGHNESS (Ks) DETERMINATION FOR REX PARK (Storm)

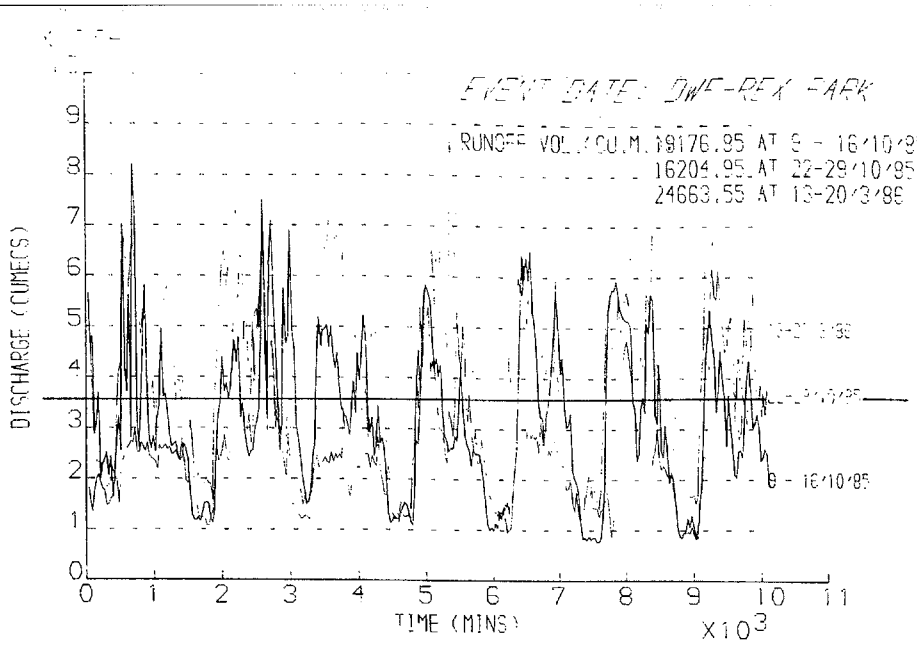


FIGURE 6.16 DRY WEATHER FLOWS AT REX PARK

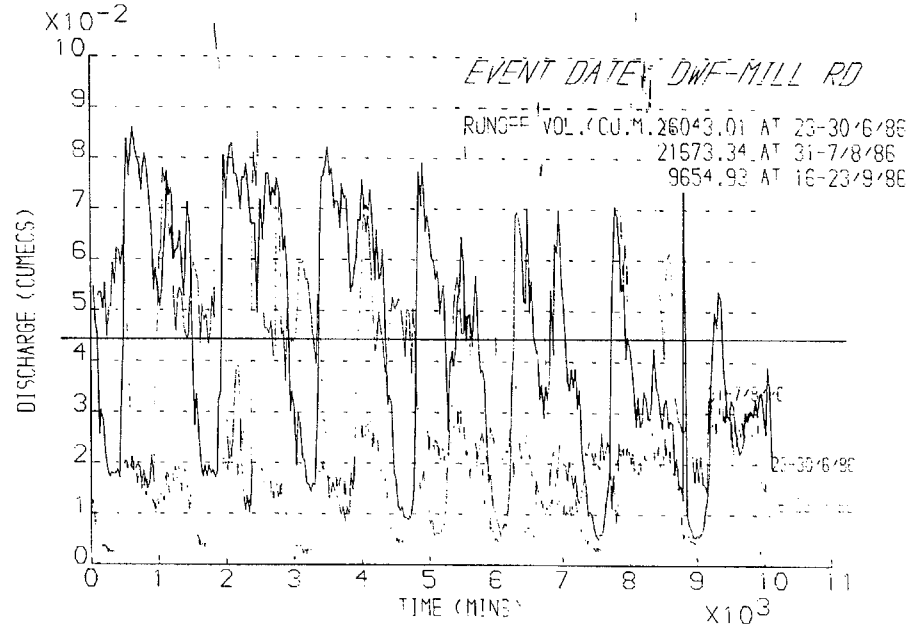
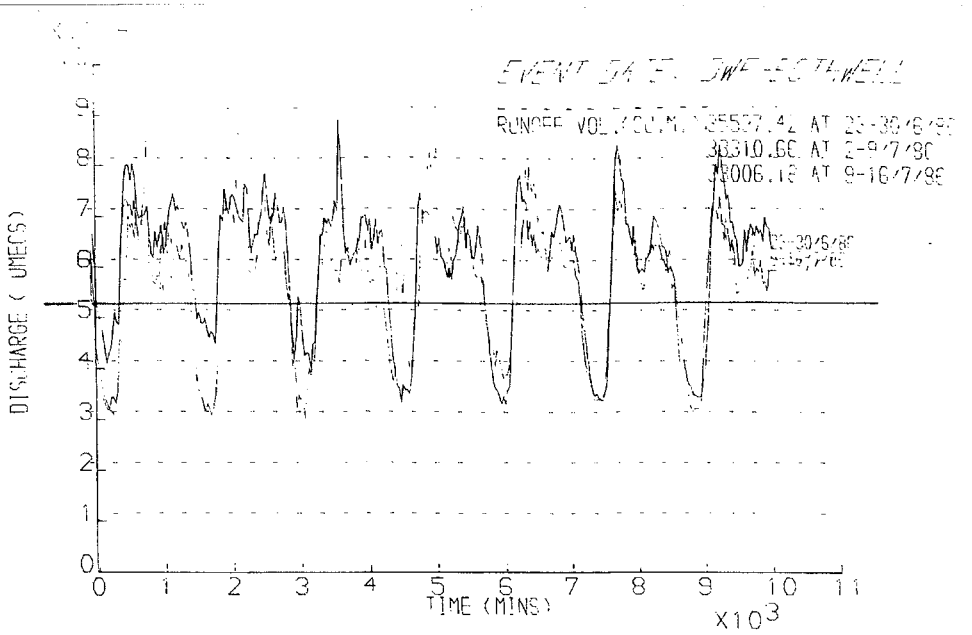
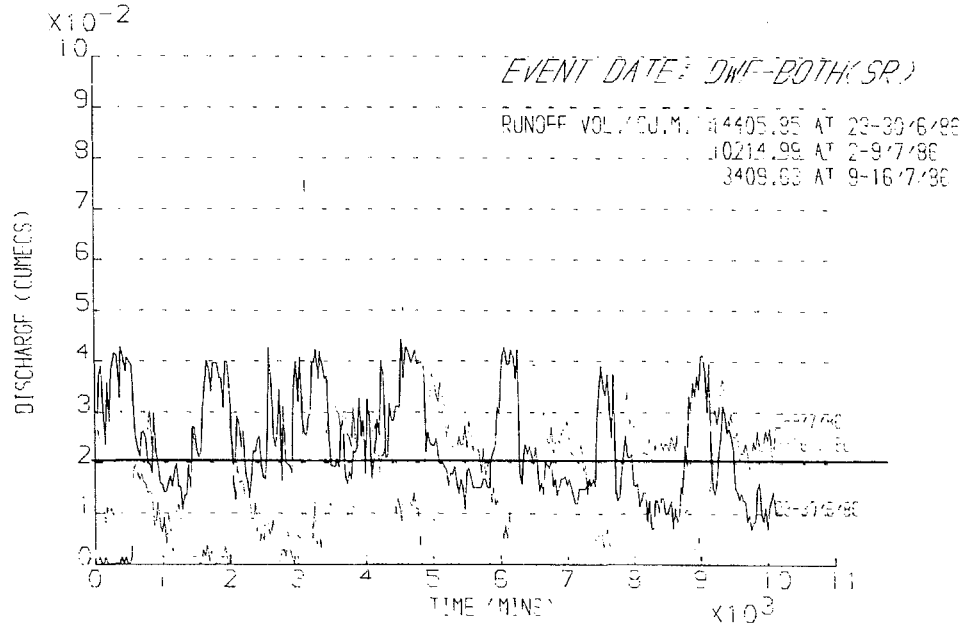


FIGURE 6.17 DRY WEATHER FLOWS AT MILL ROAD



(a) Foul



(b) Storm Relief

FIGURE 6.18 DRY WEATHER FLOWS AT BOTHWELL STREET CHAMBER

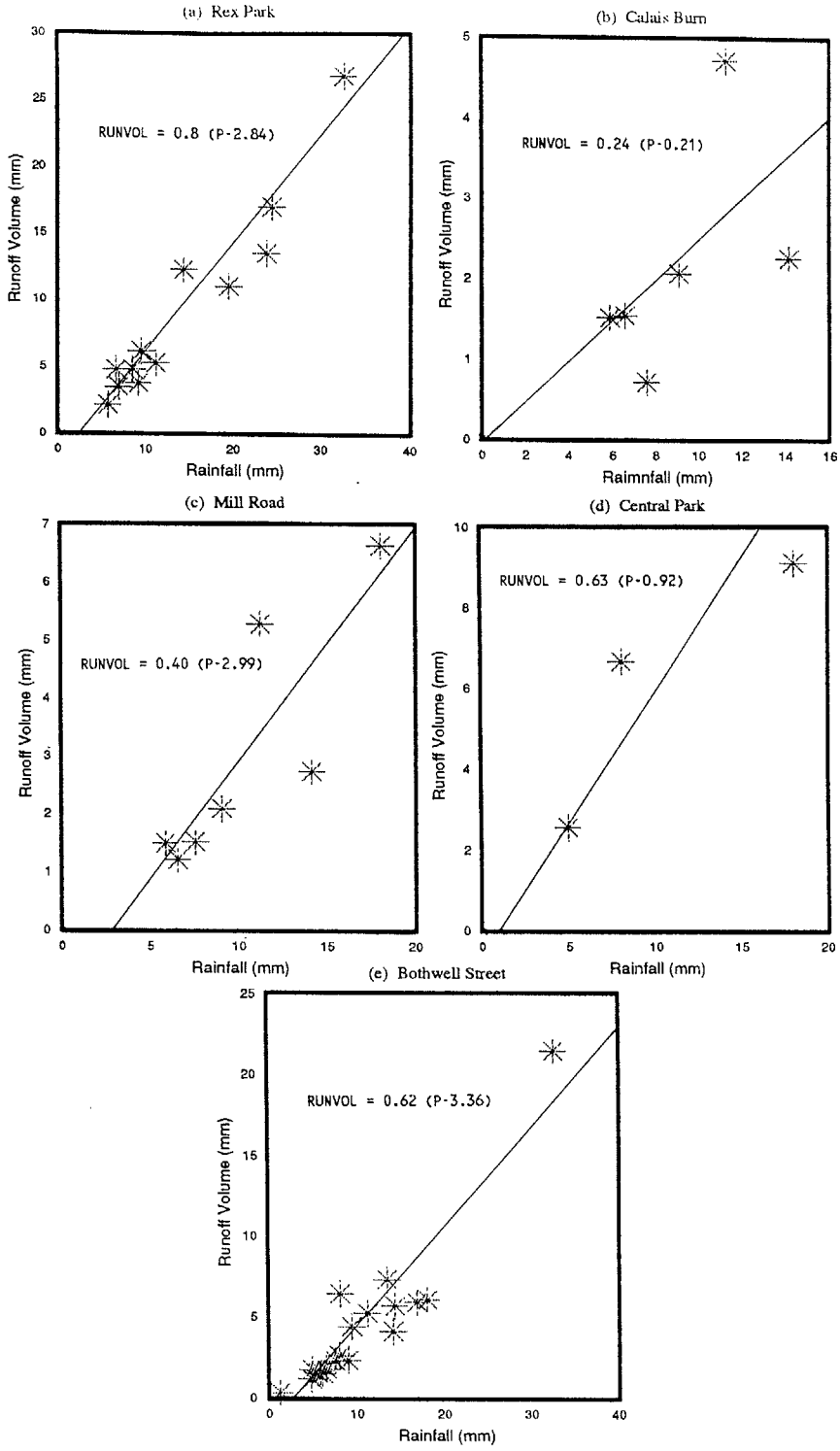


FIGURE 6.19 RUNOFF VOLUMES AGAINST RAINFALLS FOR THE LYNEBURN CATCHMENT AT VARIOUS LOCATIONS

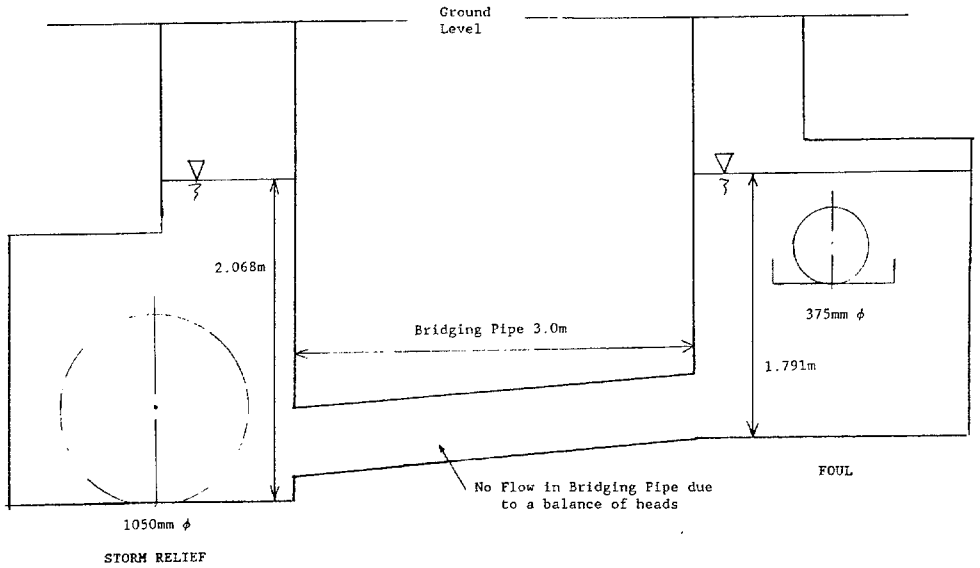


FIGURE 6.20 WATER DEPTHS IN MANHOLES AT CROSS-CONNECTION 5 DUE TO A STORM WITH 32.6mm RAINFALL ON 17 JUNE 1986



PLATE 6.1 Complete Set-Up of Dynamic Calibration for the Lambrecht 0.1mm Tipping Bucket Raingauge

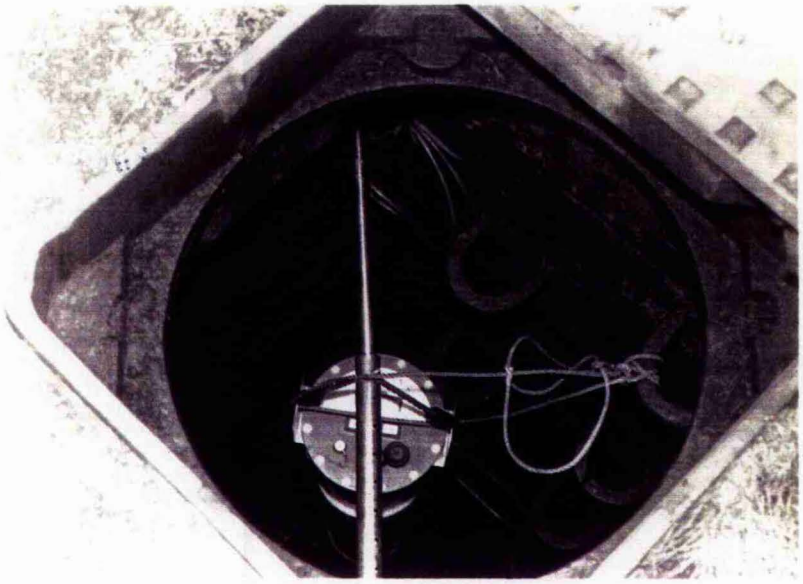


PLATE 6.2 A DET Flow Survey Logger Being Hung just Underneath the Manhole Cover to Avoid Surcharge

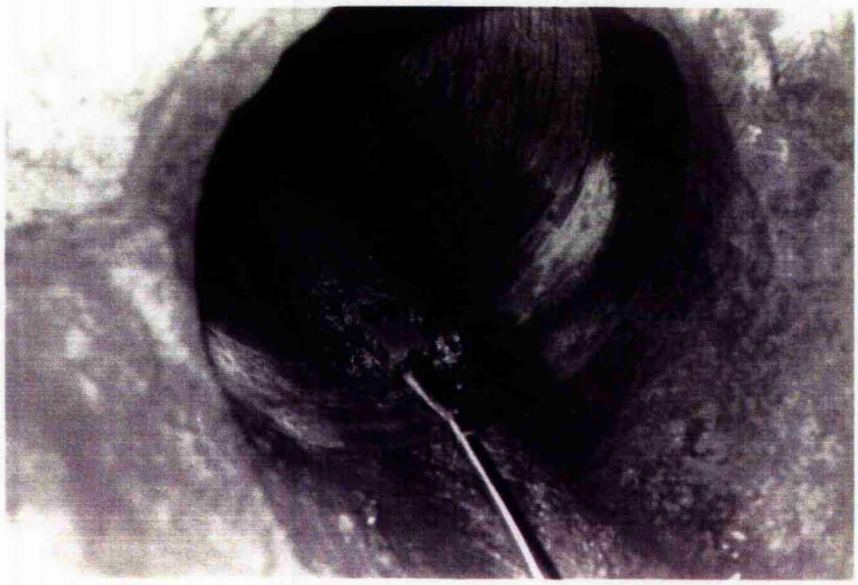


PLATE 6.3 An Ultrasonic Sensor (Mouse) Fixed on a Steel Band was Installed in the Invert of a Sewer

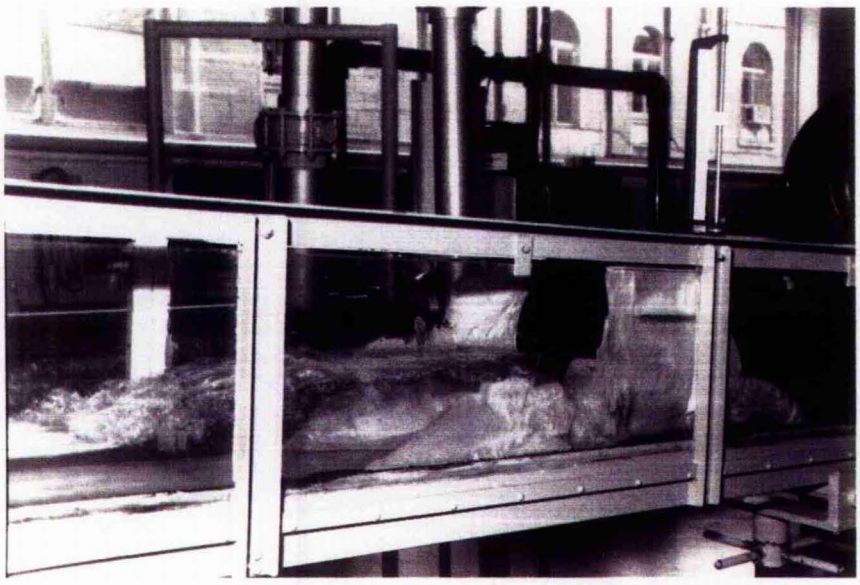


PLATE 6.4 Complete Set-Up for DET Logger Free Surface Calibration, Entrance was Tapered to Avoid Turbulence in the Pipe

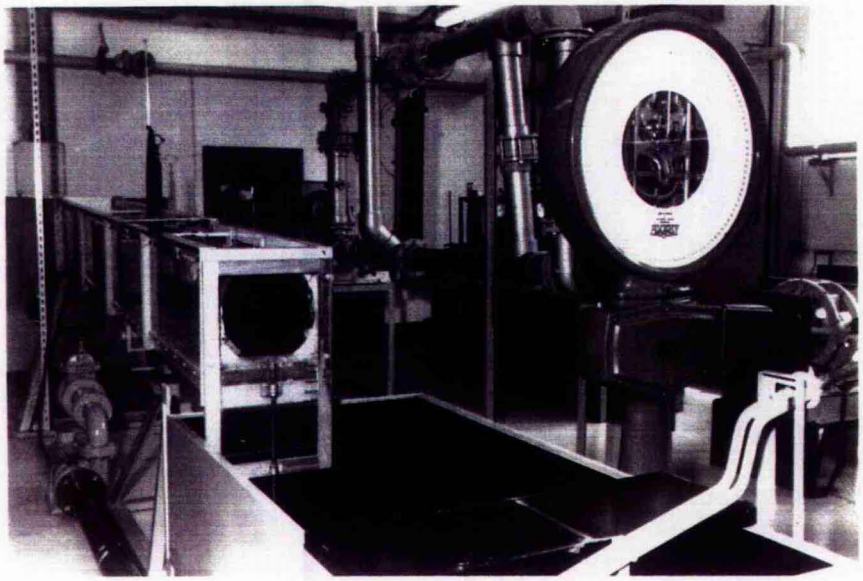


PLATE 6.5 The Exit End of the Calibration Channel and the Weighing Tank to Determine Flowrate

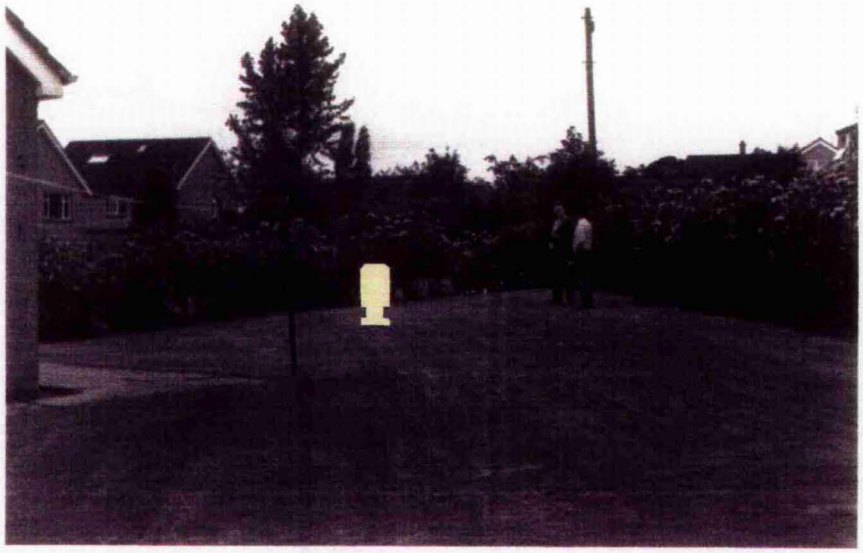


PLATE 6.6 The Raingauge Site at Scotland Drive for the Lambrecht 0.1mm Tipping Bucket Gauge

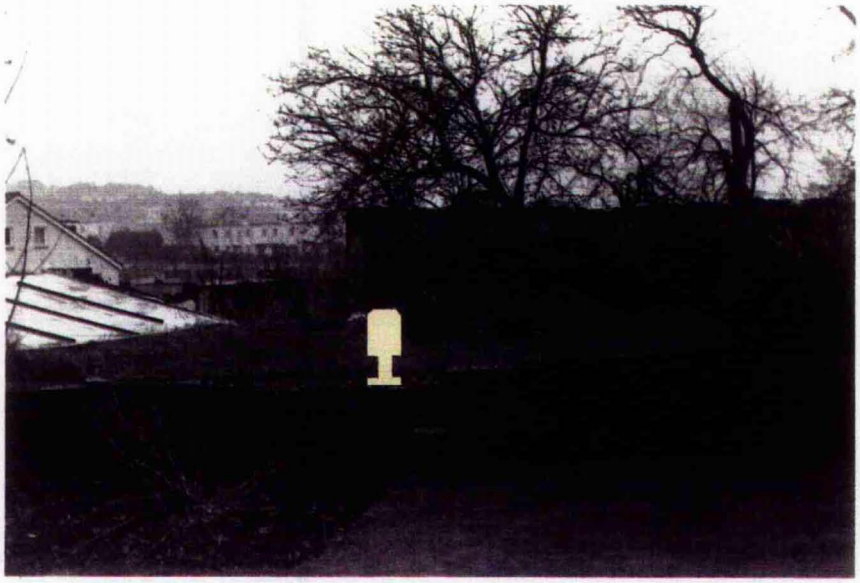


PLATE 6.7 Raingauge Site at Brucefield House for the Lambrecht 0.1mm Tipping Bucket Gauge

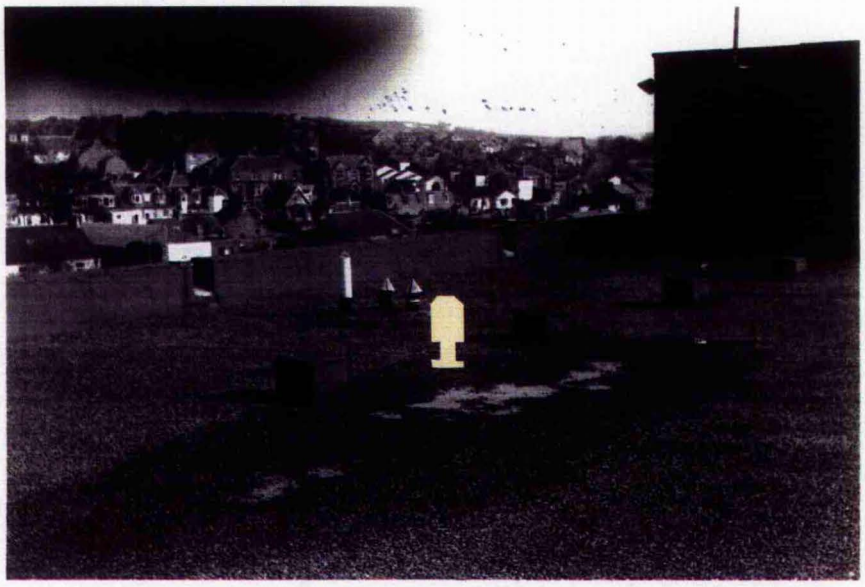


PLATE 6.8 Raingauge Site at Police Headquarter Roof for the 0.5mm IH Daily Totaliser Gauge

CHAPTER 7
SINGLE PIPE MODEL AND SYSTEM SIMPLIFICATION

7.1 INTRODUCTION

Many sewer simulation models currently exist worldwide and some of these have been discussed in Chapter 4 indicating their characteristics and general usage. Some are tailored to their study catchments and others are commercial packages usable on most dendritic sewer systems. The presence of an additional parallel pipe and of cross-connections linking the twin pipes makes the Lyneburn sewer system a complex and unconventional model for simulation. More recent simulation models, such as SPIDA (Osborne 1985) developed by Hydraulic Research Station, enable looped systems to be modelled but still cannot deal with the unusual flow behaviour in the parallel pipes and cross-connections of this system, with the need for the determination of output of flows for each of the twin pipes. Consequently a new model was required for the system.

Before an in-house computer model was constructed for the study Lyneburn parallel pipe system, it was determined that careful verification of catchment characteristics such as contributing areas; percentages of paved, pervious and roofed areas; and other overland parameters should occur. The verification process of the catchment data was achieved by contructing an equivalent combined pipe model using the simulation program in WASSP package. The details of lumping the parallel pipes into one combined pipe is presented in the following sections. Calibration and verification of this lumped pipe model did trace out a hidden contributing area at the remotest part of the Lyneburn system (Section 7.3.1). The development of the Lyneburn single pipe model and the process of identification of the hidden area have been presented at the WASSP user's group (WaPUG) meeting (Appendix G) by the author (Au-Yeung 1986). In contructing the lumped pipe model, a sewered sub-area (SSA) model was also used to simplify the large Lyneburn catchment, in order to reduce the number of pipes to the permissible 300, which the software could handle.

Very often, fully sewered catchments require to have their system simplified when a computer model is constructed. The degree of simplification is governed by one, or more, of the following criteria :

(i) Model Restrictions
The number of individual pipes or manholes is normally restricted by the computer capacity which in turn governs the number of computational routines.

(ii) Inadequate Data
Full data for both overland and below ground are not normally available for large catchments. Simplified models require less amount of input data and hence less catchment survey work.

(iii) Crude Model Construction
Often smaller or cruder models are constructed for a large catchment so that mistakes may be amended before the full model is built. Catchment data can also be verified using simplified model.

As seen from Figure 5.6, the Lyneburn catchment can be classified into two groups according to the function of the sub-systems, i.e. combined and separately draining systems. For this reason, the catchment can be neatly subdivided into sub-areas with each of these self-contained sub-systems being represented by a SSA model. Before the in-house parallel pipe model was developed, an equivalent combined pipe (or lumped pipe) model was constructed for the Lyneburn twin pipe system. The object in developing a combined pipe model was to :

(i) Verify the performance of the simplified sub-areas,
(ii) Clarify the percentages of contributing, paved and roofed areas,
(iii) Identify any possible hidden contributing areas draining directly to the Lyneburn system, and
(iv) Adapt these verified catchment data for the future parallel pipe model.

Two versions of lumped pipe model have been constructed. In addition to the WASSP model which was in the micro-computer, the

second one used DUCTS single pipe model (Ashley & Jefferies 1984, Jefferies & Ashley 1984) developed in the main-frame computer. The performance of these two models were then assessed in Section 7.3.

7.2 STUDY CATCHMENT SIMPLIFICATION

There are a number of procedures for simplifying catchments and the performance of the methods have been discussed by previous researchers (Price et al 1980, Catterson 1986, Nussey 1986). Most of the alternative methods of catchment simplification range from minor reduction of the number of pipes down to the one-piped Sewered Sub-Area model (SSA). In order to adopt the most suitable method for the Lyneburn system, two self-contained sub-catchments were chosen for comparison of the different simplification procedures.

A brief overland survey was carried out for the global catchment in order to categorise the typical types of sub-areas according to their topographical features and housing type. This survey was initially based on the record plans and was followed by close examination of the catchment. It was found that all sub-areas within the catchment could be represented by only two subcatchment types. The first is the housing type having high permeable area but low paved and roofed areas, whilst the second is the multi-storey housing with high paved and roofed area but low in permeable area. One from each of these two sub-catchment types was then used for the testing of the simplification procedures. A rain-gauge and a flow data-logger were installed on one of the sub-catchments, and simulation outputs for a full model and various simplified models were compared with the observed data.

Once the simplification method was chosen, the procedure was used extensively for the entire catchment to minimise the number of nodes and pipes in the final model. The number of pipes on the main branches was not reduced in order that the actual flow behaviour in the trunk sewers could be maintained.

7.2.1 SUB-CATCHMENTS FOR SIMPLIFICATION

The two chosen sub-catchments, Scotland Drive and Garvock Bank, are similar in size and have the same outfall pipe diameters. Their locations in the Lyneburn catchment has already been shown in Chapter 5 (Figure 5.5). Figures 7.1 and 7.2 show each system layout and housing details of the two sub-catchments and Table 7.1 summarises the catchment characteristics of both. Scotland Drive is a steep catchment with low density housing and a substantial amount of permeable area. Garvock Bank, in contrast, is an area consisting of flat-roofed multi-storey council flats having a high percentage of paved areas.

Rainfall and outflow data were monitored for the Scotland Drive sub-catchment only. The locations of the rain-gauge and flow data-logger are shown in Figure 7.1. Due to various problems during the three months of data collection, only one usable event was monitored successfully by both rainfall and flow loggers. Normally three events are required to demonstrate the performance of a model (Williams 1984[1]). It was felt in this case that one event was sufficient to demonstrate model simplification and performance. The Garvock Bank sub-catchment was not gauged and could not be used for model testing. However, the survey data and the experience gained from the model constructed was used for other similar sub-catchments in the system.

Simplified models using WASSP and DUCTS were constructed for both sub-catchments. Full single pipe models were then assembled using the chosen simplified sub-model for Lyneburn sewer system on WASSP and DUCTS.

7.2.2 COMPARISON OF SIMPLIFIED MODELS

In addition to the complete model, three different simplified models were constructed for Scotland Drive.

The models tested were :

(i) Five-pipe
(ii) Three-pipe
(iii) One-pipe Sewered Sub-Area (SSA)
(iv) Full model

The above simplifications are shown diagrammatically in Figure 7.3 for the Scotland Drive sub-catchment.

Figure 7.4 shows all four output hydrographs from WASSP simulations plotted against the observed data for the event of 30/9/85. All the results obtained from the various simplified models were compared with the full model and the observed data. The maximum difference between the highest and lowest peaks was approximately 12%. Figure 7.4 also demonstrates that the SSA model produced a closer fit to the observed hydrograph than the other cruder simplification methods.

The SSA option in DUCTS was utilised in the similar exercise for the Scotland Drive sub-catchment. Results are again plotted in the hydrograph form and compared with the WASSP sewered sub-area model and shown in Figure 7.5. It can be seen that both models give very similar outputs for the peak flows, runoff volume and timing. The comparison also suggests that the SSA model in WASSP and DUCTS are predicting very close results.

Two simplified models were constructed for Garvock Bank using the SSA method of both WASSP and DUCTS. Results are plotted in Figure 7.6 for the DUCTS model against the full WASSP model for Garvock Bank. The maximum difference between the peaks was about 15%. The greater difference in the peaks may well have been due to the much higher percentage impermability of the Garvock Bank sub-catchment resulting in higher percentage runoff (PR) and greater discrepancies. Despite the high percentage difference in the peak, the hydrographs were close to each other, especially at the beginning of the event. Furthermore, the highest peaks between WASSP and DUCTS were still close and all were within the ±20% (Price & Osborne 1986).

7.2.3 ADOPTION OF SEWERED SUB-AREA MODEL

Comparison of the various models showed that the SSA model could be used for catchment simplification. The close fits between the observed data and the models suggested that reducing the number of pipes did not substantially affect the flows in the system, particularly the peaks. In fact, as shown in the development of the SSA model, one catchment including 109 pipes was represented satisfactorily using this method (Price et al 1980). However, there are limitations on the use of sewered sub-areas model, these being :

(i) Catchment size should not be too large :
As suggested by the HRS (1981), SSA model may only be used on sub-areas up to 60 ha in area. Little loss of accuracy will result.

(ii) No surcharge should be allowed :
Flow routing procedures in the SSA model utilise the Muskingum-Cunge method which is for free surface flow, but inadequate for pressurised condition (Price et al 1980).

(iii) Volume is not adequately modelled :
Again due to the lack of pressurised flow routing in SSA model, flow storage in manholes is not capable to be modelled.

(iv) Accurate data inputs :
Data such as total length of the major pipe, average pipe slope and the diameter of the outfall pipe require to be accurately input. Sensitivity analyses have indicated that a 10% error in estimating any one of these would result in a maximum error in peak discharge of 3% (HRS 1981).

The SSA model was applied to the Lyneburn sewer system. A brief overland survey was again carried out, in this case only to justify the boundaries for the self-contained sub-areas in the catchment based on the topographical features. Each area contributing to the parallel pipes was then represented by a SSA model. A global WASSP model for the entire catchment was completed within the permissible number of pipes and nodes, which was 300 for the version of micro-

WASSP used (HRS 1984). The sub-areas subjected to the simplification process have been shown in Chapter 5 (Figure 5.6) and they formed a series of inflows to the parallel pipe system. The DUCTS lumped pipe model was also constructed for the study system. Even though DUCTS was developed on the main-frame computer and did not have the restrictions of WASSP, simplified models were retained to save time and because they were convenient in the various reconstructions of the model.

7.3 EQUIVALENT SINGLE PIPE MODEL

To gain an understanding of survey data and catchment parameters, such as percentage runoff and depression storage, WASSP and DUCTS models were firstly constructed for the full Lyneburn catchment and then used to obtain reasonable fits between observed and simulated outputs.

To carry out this exercise, the twin pipes were lumped to form a single equivalent pipe model. The combination of the parallel pipes was based on the assumption that the friction head loss in each pipe was constant (Ashley & Jefferies 1984). In addition, it was assumed that the two pipes had similar hydraulic resistance characteristics. The methodology and two examples of the procedure used to combine the two pipes into a single equivalent are shown in Appendix D.

Single pipe models were constructed for two locations where observed data were available :

(i) Rex Park
(ii) Bothwell Street overflow chamber

Table 7.2 lists the events available for simulation. In addition, models were also constructed for three further areas :

(iii) Calais Burn
(iv) Mill Road
(v) Central Park

All the above locations and their boundaries have been mentioned in Chapter 5 and shown in Figure 5.8. Table 5.2 also gives their sizes of contributing areas.

7.3.1 ADOPTION OF WASSP

WASSP models were constructed for the above five catchments. However, some main sewers required further simplification in the Rex Park, Mill Road and Bothwell Street models because the total number of pipes exceeded 300. This exercise of combining several short lengths of pipes into a single one was only carried out for those having similar pipe gradients and diameters (Jefferies et al 1986). During simplification, the combined pipe required to have its overall length checked for being not less than 10 times the pipe diameter in order not to lose the routing accuracy. Furthermore, actual numbers of manholes in pipes being combined were input as optional, so that head loss could be accounted for during surcharge.

Model calibration for the study system using WASSP showed that one hidden contributing area existed. This was the Crossgates sub-catchment. This area is situated at the remotest location of the Lyneburn catchment and storm flows only enter the main sewer when rainfall exceeds approximately 15mm. The identification of this hidden area has been presented elsewhere (Au-Yeung et al 1986) and a copy is included as Appendix G.

Simulations were performed using events listed in Table 7.2. Results and comparisons are included in section 7.4.

7.3.2 ADOPTION OF DUCTS

Unlike the version of micro-WASSP used, there was no pipe number limitation on DUCTS and hence the actual number of sewers used was as in the system i.e. 389 at Bothwell Street overflow chamber.

Since SSA models in WASSP and DUCTS were predicting close results, all the self-contained contributing sub-areas as represented by SSA in WASSP were retained to be the same in DUCTS.

Again unlike WASSP, the SSAs in DUCTS could only be input at the top of a branch. Those sub-areas as in the middle of a branch in WASSP had therefore become a new branch in DUCTS. Despite the change in the SSA input format in the two models, the total number of sewered sub-areas remained the same. As it was on the mainframe computer, DUCTS had no limitations in the number of active branches in the system and therefore would not affect the flow simulation and the outputs.

The events used for simulation are listed in Table 7.2. The results were plotted as hydrographs and compared with both the observed data and the WASSP simulation outputs.

7.4 COMPARISON OF RESULTS

Simulation outputs for the five WASSP models have been plotted and compared with the combined observed hydrographs for the parallel pipes. Figures 7.7 to 7.10 show the hydrographs for the different models while Table 7.3 summarises the peak discharges, and runoff volumes.

A similar exercise was carried out for the DUCTS simulation outputs for the different models. The hydrographs are shown and plotted not only against the observed data, but also in comparison with the WASSP output in Figures 7.11 to 7.15. Table 7.4 again lists all peak discharges and runoff volumes for both WASSP and DUCTS results.

The computed results for most of the models generally show slightly greater peak flows than observed, but indicate in general very close fits with the observed data. A longer flow data logging duration was established for the Rex Park and Bothwell Street catchments and this resulted in wider choice of the data logging time intervals which was governed by the flow level during storms.

The complete monitored flow data for most events for these two sites gave better match between observed and predicted hydrographs and hence results were best of all. The Calais Burn model, in contrast, shows a poorer fit as compared with the observed values. This was primarily attributed to the fact that the data-logger did not trigger and kept capturing data at a 30 minute time interval for one storm only and hence details of the flow data during the storm were missed completely.

The peak discharges listed in Tables 7.3 and 7.4 were plotted with the computed peaks against observed peaks. Figure 7.16 shows the comparison of peak flows for WASSP models and observed data while Figure 7.17 shows all the DUCTS, WASSP and observed peaks. A very good linear fit can be observed from the figures and all plotted points lie within the ±20% band.

It can be seen from Tables 7.3 and 7.4, and Figures 7.16 and 7.17 that all the flow peaks and runoff volumes are within the acceptable percentage region as suggested (Price & Osborne 1986). It can be concluded that the simulations using the lumped model show that all the models performed satisfactorily. The close fit between the WASSP and DUCTS also suggested that the survey data used was robust and hence the following may be used in the parallel pipe model development :

(i) The simplified model for the contributing areas;
(ii) Catchment data such as contributing areas, percentages of paved, pervious and roofed areas;
(iii) pipe gradients, roughness values and other pipe details for the parallel pipes remain unaltered;
(iv) All lateral inflows remain unchanged Percentage Runoff (PR) (as those derived in Chapter 6) will be used for the parallel pipe model verification.

SUBCATCHMENT DETAILS	SUBCATCHMENT LOCATIONS	
	SCOTLAND DRIVE	GARVOCK BANK
Description of Subcatchment	Modern Semi-Detached or Detached Estate	Mulit-Storey Council Flats
Total Sub-Catchment Area	11.892 ha	9.994 ha
Percentage of Impervious Area (PIMP)	14	58
Percentage of Roofed Area (PRroof)	10	11
Percentage of Flooded Area (PRflood)	5	5
Paved Area Per Gulleys (PAPG)	Less than 200 m^2	Between 200 and 400 m^2
Average sub-Catchment slope (SLOPE)	0.0524	0.0340

TABLE 7.1 SUBCATCHMENT DETAILS FOR SCOTLAND DRIVE AND GARVOCK BANK

EVENT	RAINFALL DEPTH (MM)	SUBCATCHMENT WHERE MODEL HAS BEEN BUILT				
		RP‡	CB‡	MR‡	CP‡	BS‡
860517	11.2	WASSP				
860617	32.6					WASSP
860806	9.1		WASSP/ DUCTS	WASSP/ DUCTS		WASSP/ DUCTS
860813	5.9		WASSP	WASSP		
860902	18.1			WASSP/ DUCTS	WASSP/ DUCTS	

‡ RP -- Rex Park
CB -- Calaisburn
MR -- Mill Road
CP -- Central Park
BS -- Bothwell Street

TABLE 7.2 OBSERVED EVENTS AND SIMULATION MODELS CHOSEN FOR THE FIVE CATCHMENTS

LOCATION	EVENT	PEAK FLOWRATE (m^3/s)		RUNOFF VOLUME (m^3)	
		Observed	WASSP	Observed	WASSP
Calaisburn	860813	0.206	0.263	1434.6	1455.7
Rex Park	860517	0.617	0.690	7658.2	7430.9
Mill Road	860813	0.658	0.666	4300.6	4608.5
Bothwell Street	860617	4.329 3.924 2.311	4.282 3.937 2.328	50144.5	47999.1

TABLE 7.3 PEAK DISCHARGES AND RUNOFF VOLUMES OF WASSP AND OBSERVED VALUES

LOCATION	EVENT	PEAK FLOWRATE (m^3/s)			RUNOFF VOLUME (m^3)		
		Obs	DUCTS	WASSP	Obs	DUCTS	WASSP
Calaisburn	860806	0.181	0.180	0.191	1555.1	1612.0	1861.0
Mill Road	860806	0.574	0.554	0.583	6426.0	7589.9	6452.5
	860902	1.076	1.072	1.066	12067.9	12770.4	10105.2
		0.491	0.461	0.478			
		0.861	0.813				
Central Park	860902	0.231	0.232	0.252	6329.5	5768.1	4500.8
		0.373	0.333	0.351			
		0.308	0.302				
Bothwell Street	860806	0.857	0.889	0.906	11515.1	13049.8	11892.1

TABLE 7.4 PEAK FLOWS AND RUNOFF VOLUMES FOR ALL MODELS

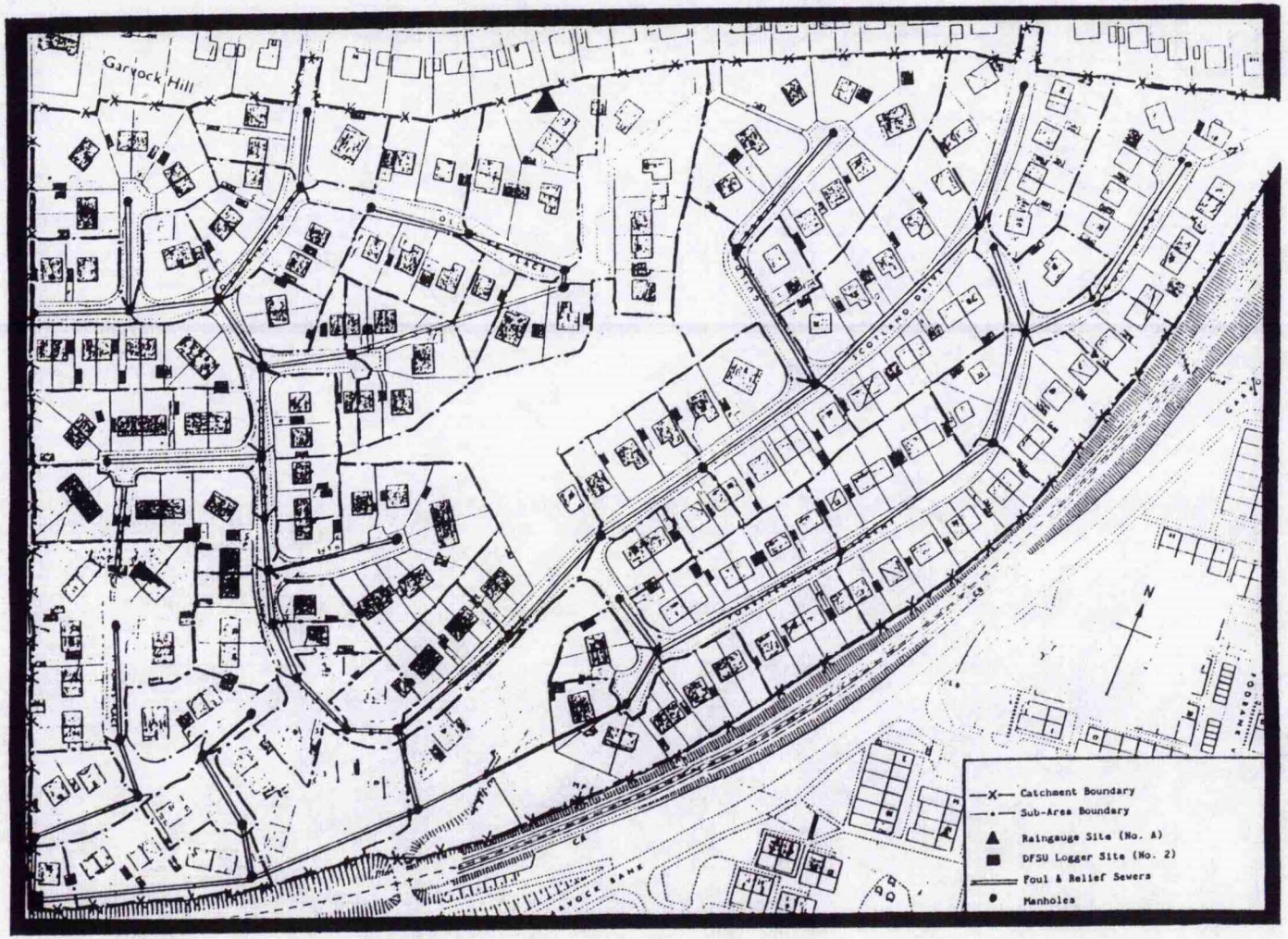


FIGURE 7.1 Scotland Drive Sub-Catchment

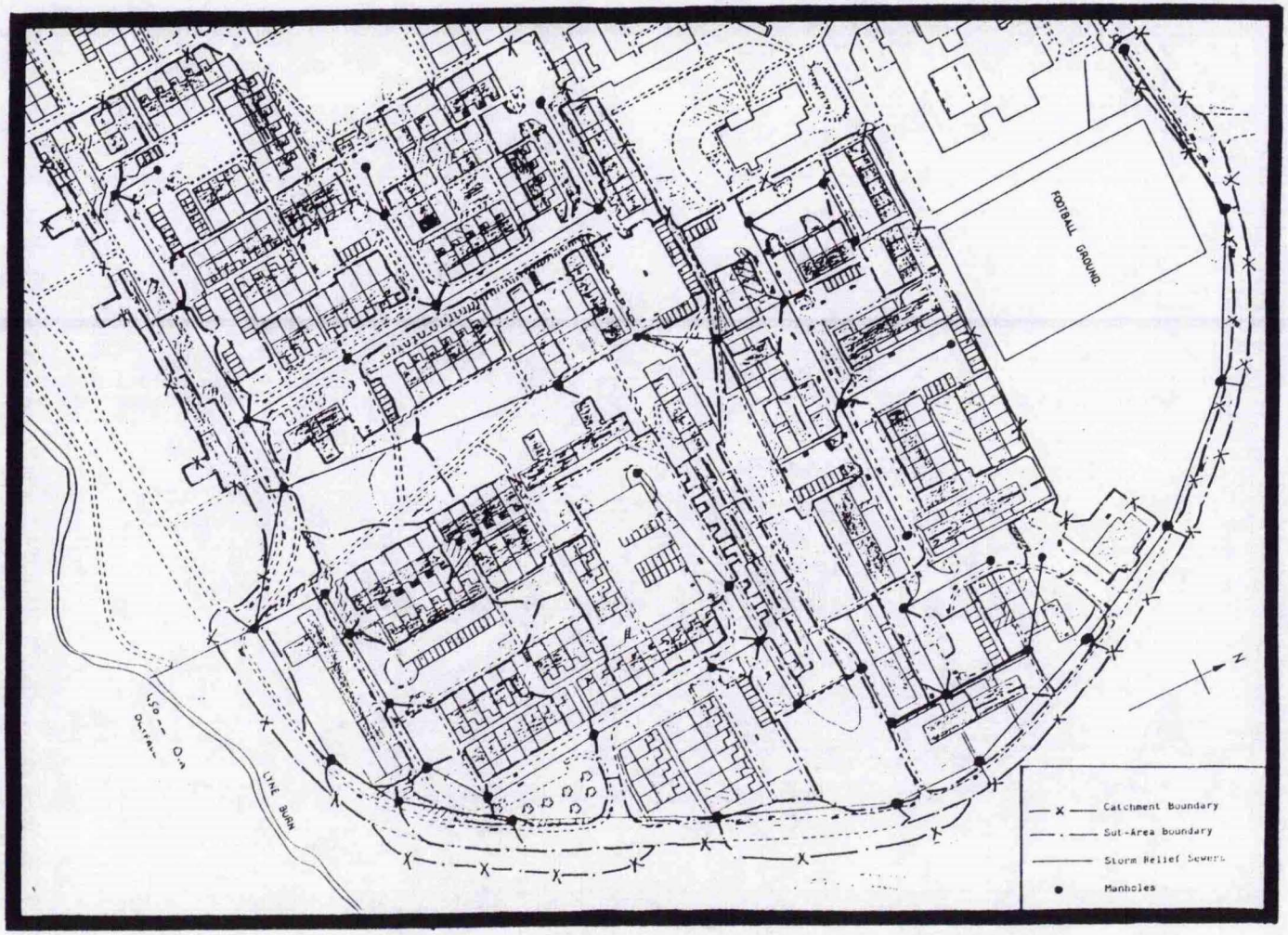


FIGURE 7.2 Garvock Bank Sub-Catchment

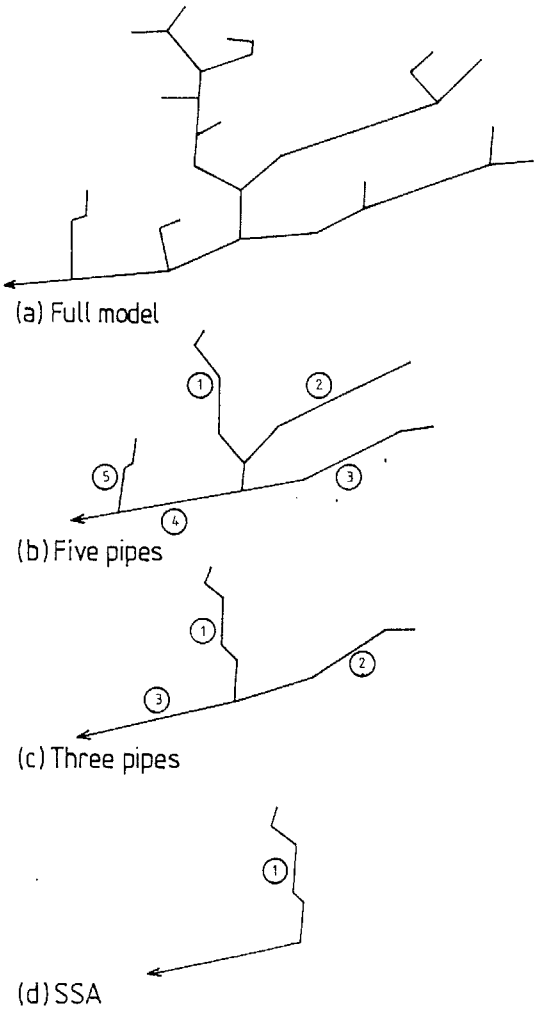


FIGURE 7.3 Diagrammatic Sketch of Full and Simplified Models for Scotland Drive Sub-Catchment

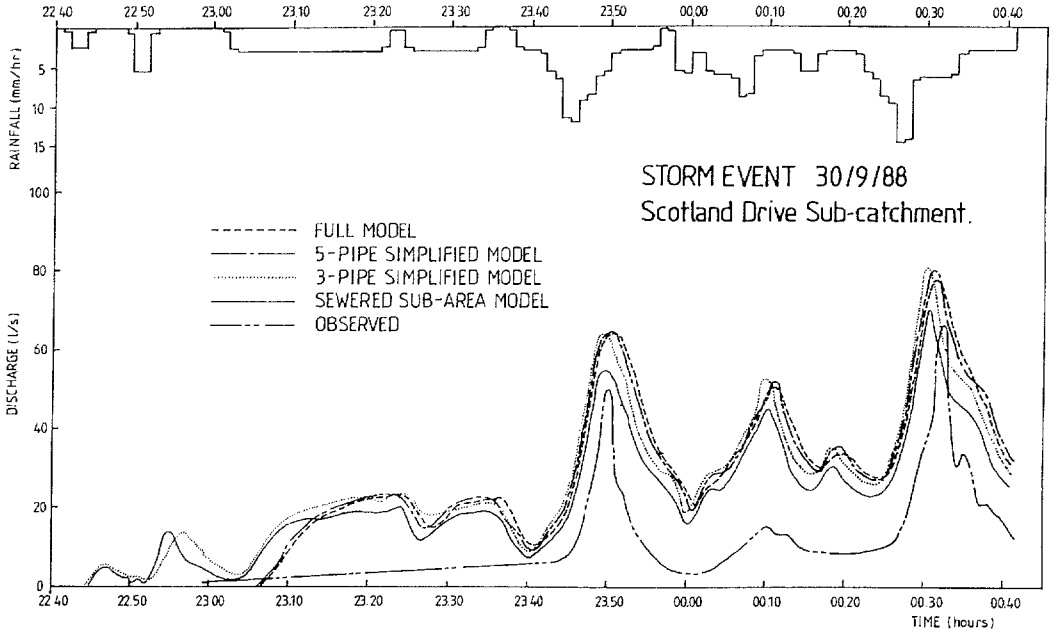


FIGURE 7.4 Comparisons Among Simplified and Full Simulation Model with the Observed Flow at the Outfall

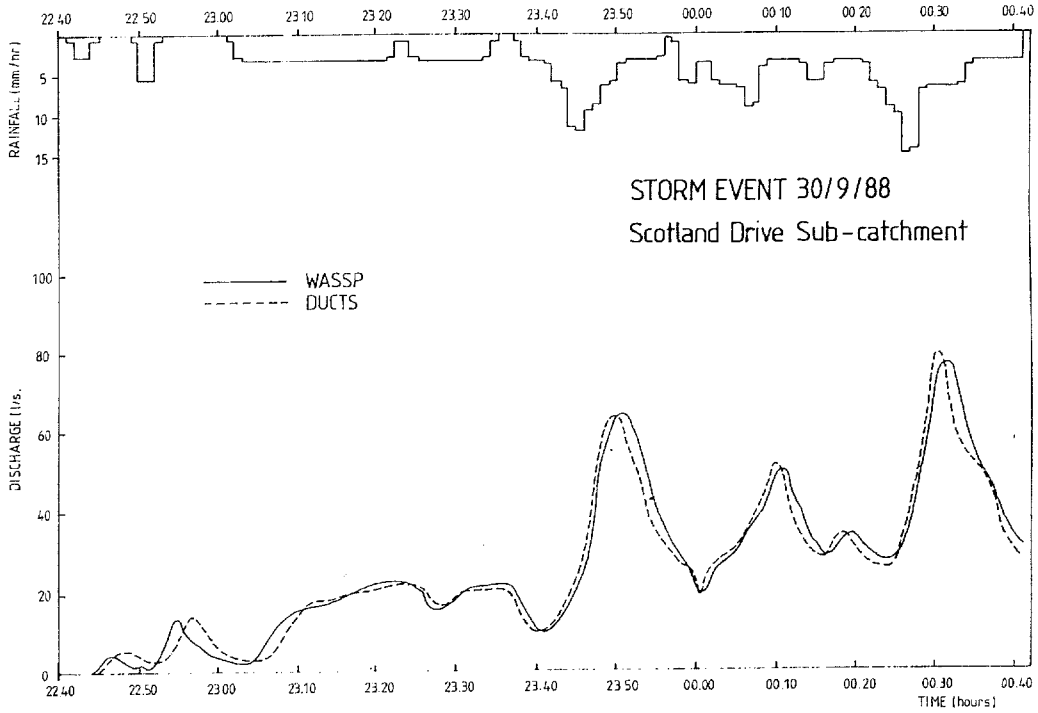


FIGURE 7.5 Comparison between WASSP and DUCTS simulation Outputs at Scotland Drive Sub-Catchment Outfall

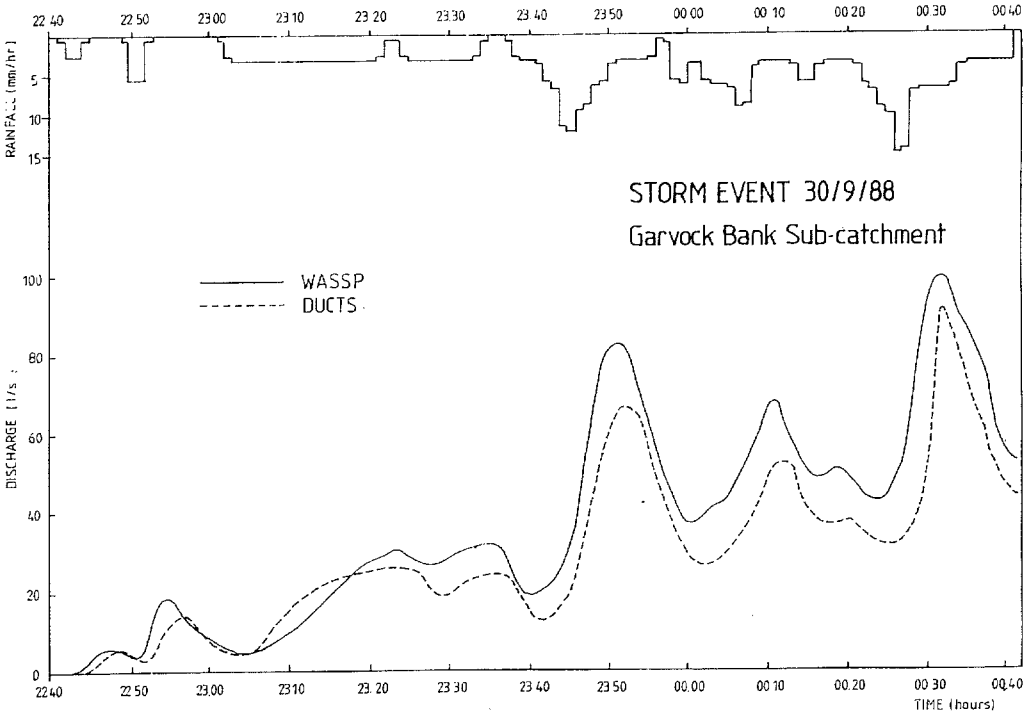


FIGURE 7.6 Comparison between WASSP and DUCTS simulation Outputs at Garvock Bank Sub-Catchment Outfall

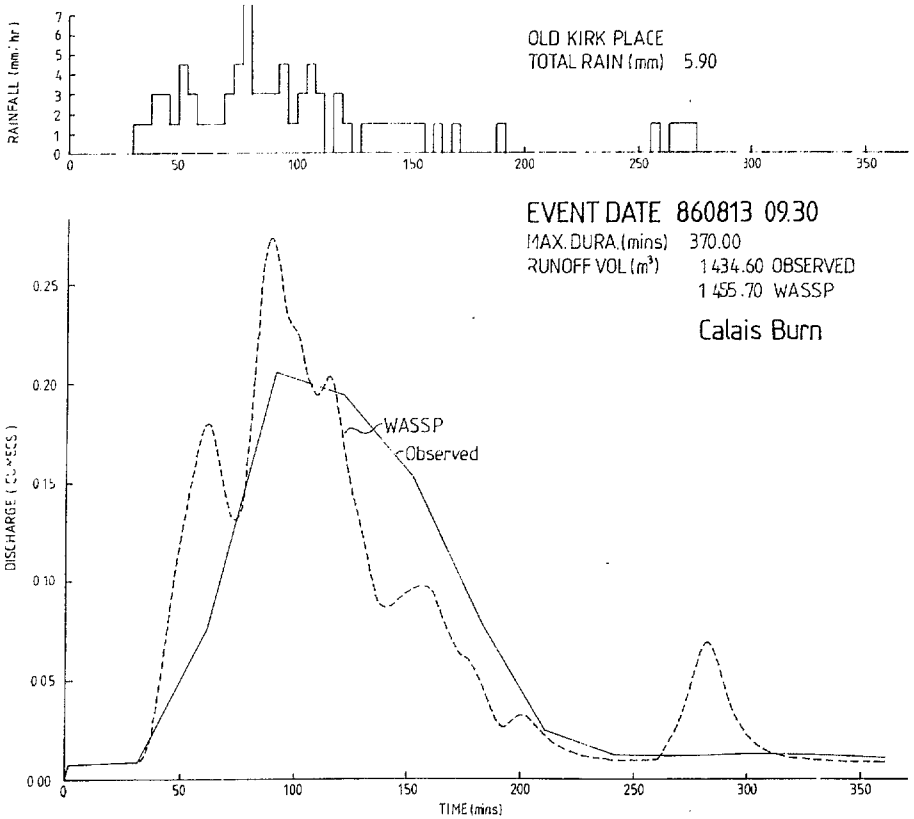


FIGURE 7.7 Comparison between Observed and WASSP Computed Flows at the outfall of Calais Burn

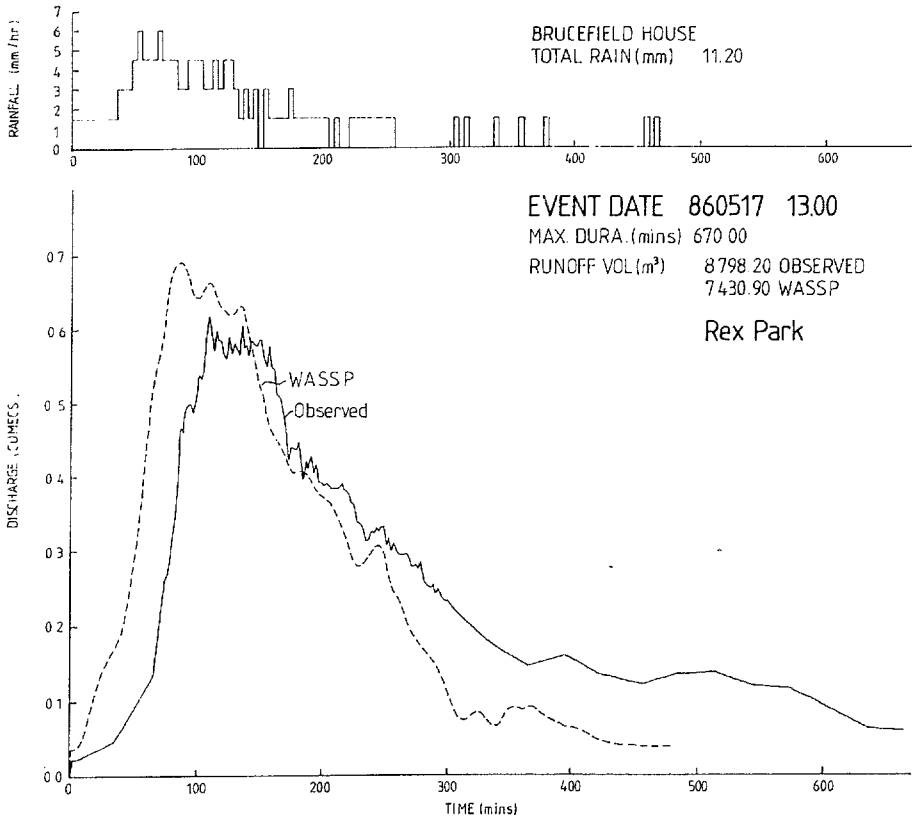


FIGURE 7.8 Comparison between Observed and WASSP Computed Flows at the outfall of Rex Park

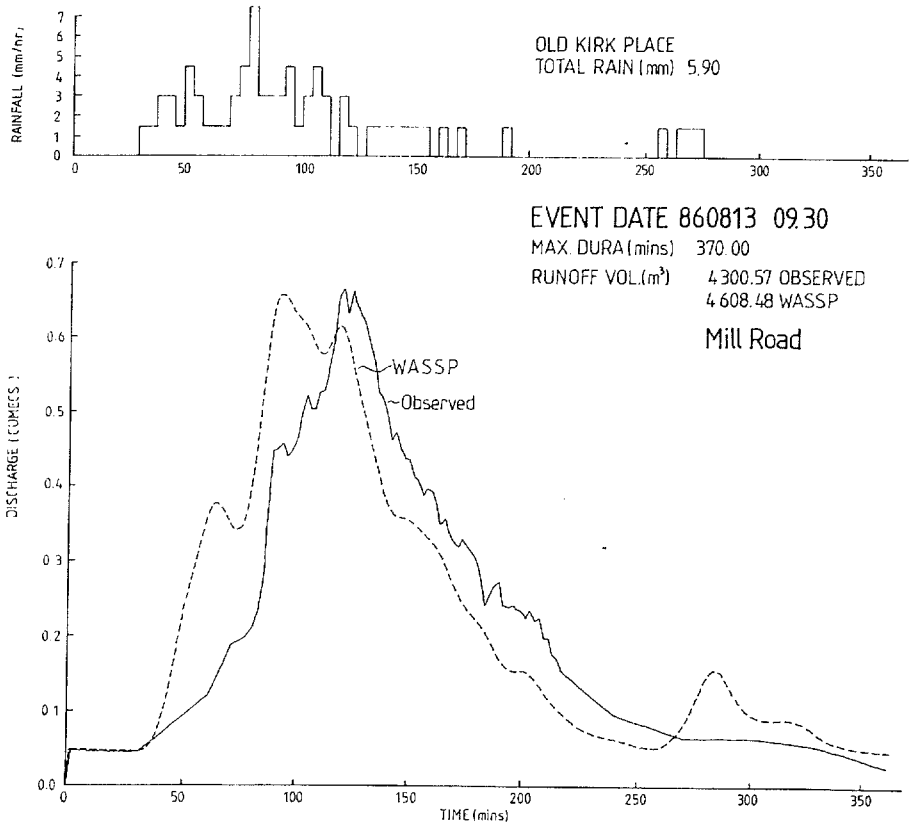


FIGURE 7.9 Comparison between Observed and WASSP Computed Flows at the outfall of Mill Road

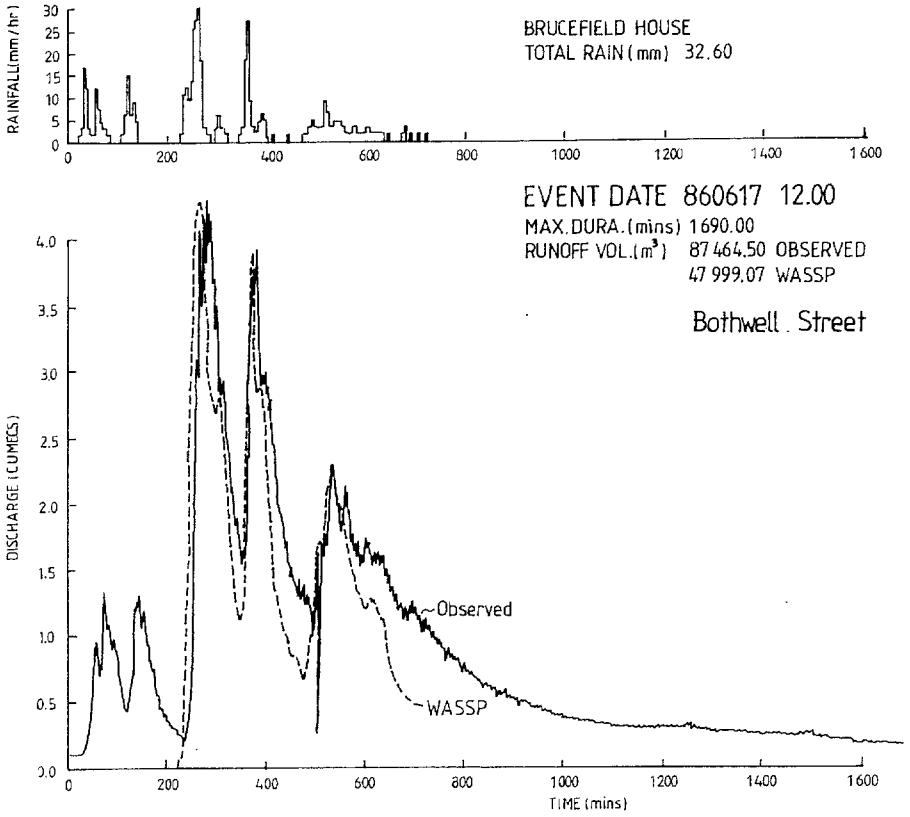


FIGURE 7.10 Comparison between Observed and WASSP Computed Flows at the outfall of Bothwell Street Chamber

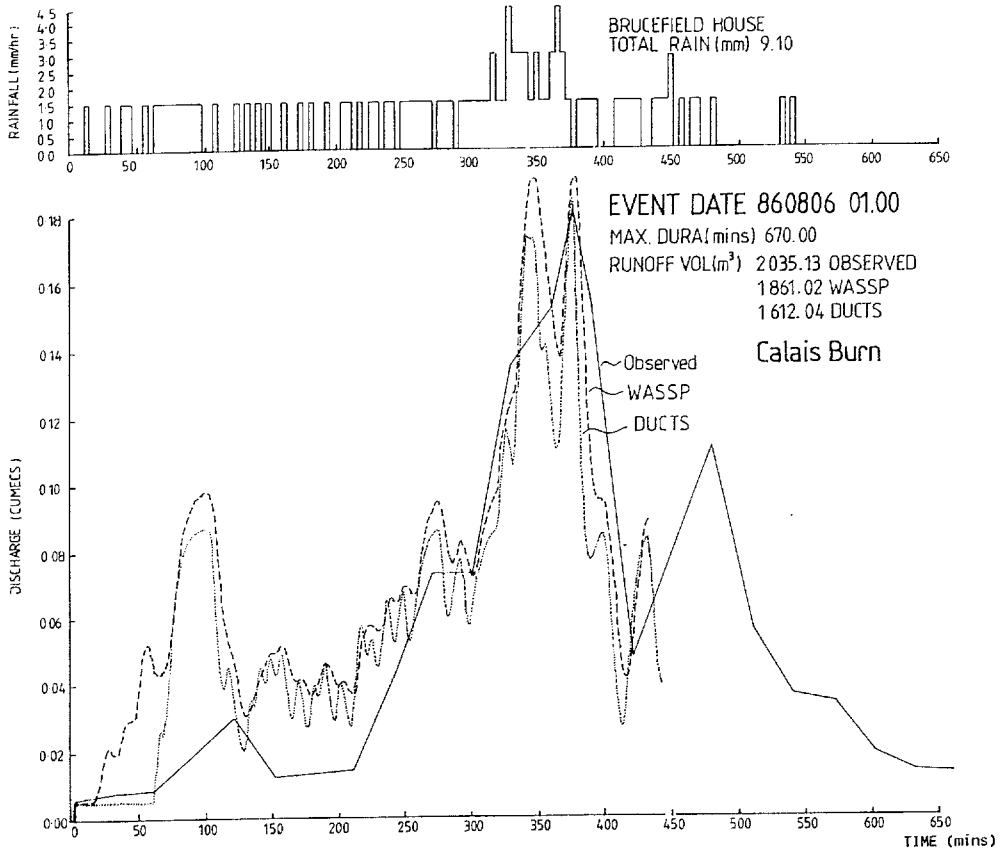


FIGURE 7.11 Comparison between Observed and WASSP and DUCTS Computed Flows at the outfall of Calais Burn

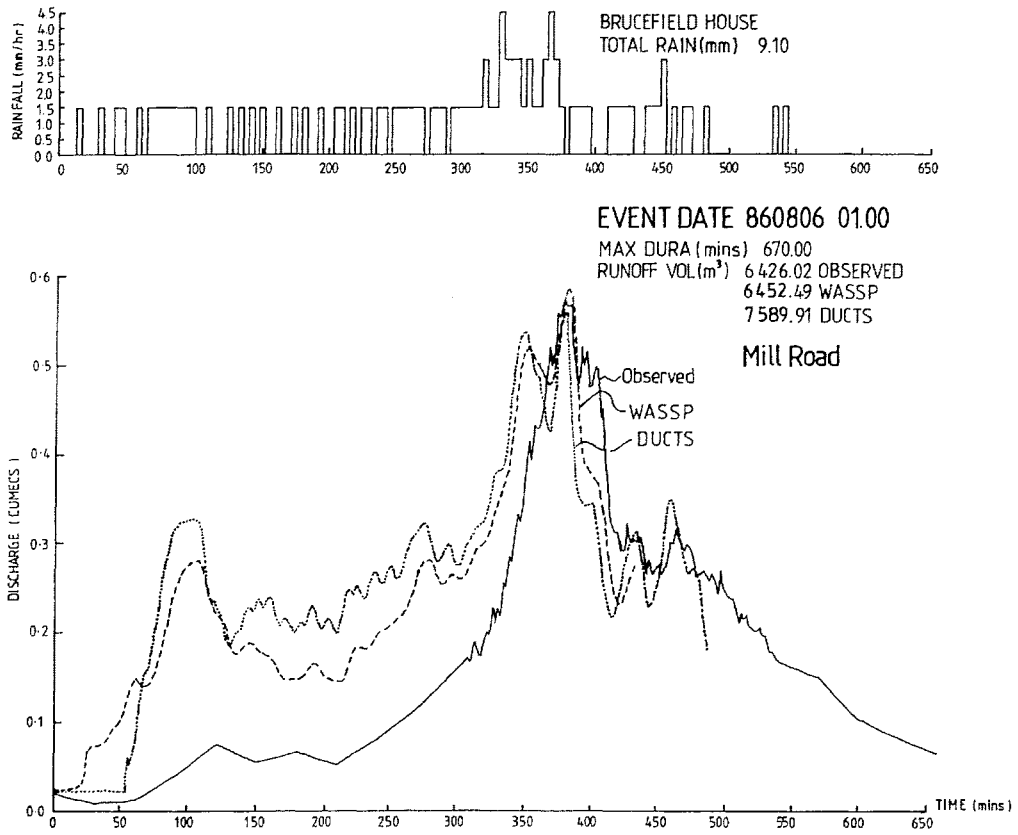


FIGURE 7.12 Comparison between Observed and WASSP and DUCTS Computed Flows at the outfall of Mill Road

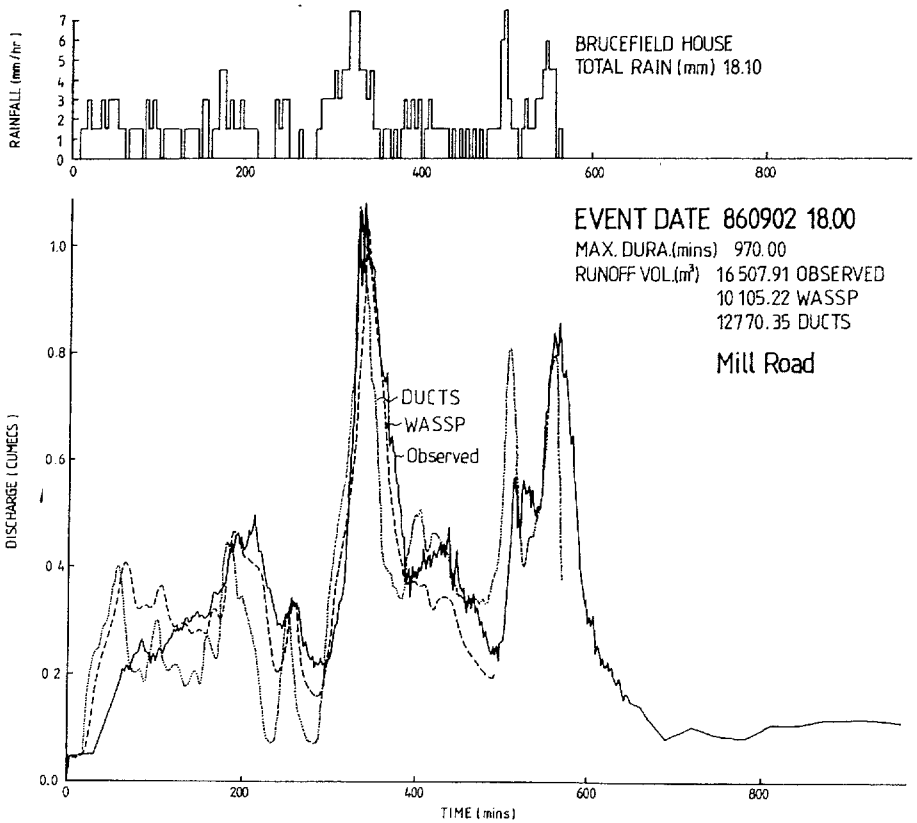


FIGURE 7.13 Comparison between Observed and WASSP and DUCTS Computed Flows at the outfall of Mill Road

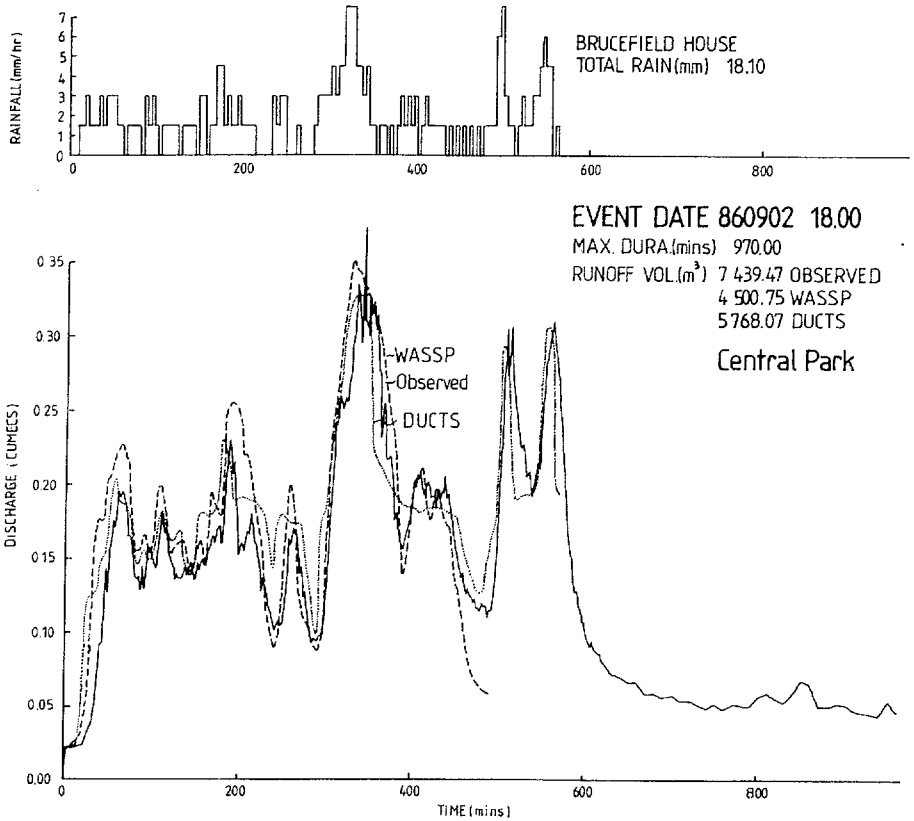


FIGURE 7.14 Comparison between Observed and WASSP and DUCTS Computed Flows at the outfall of Central Park

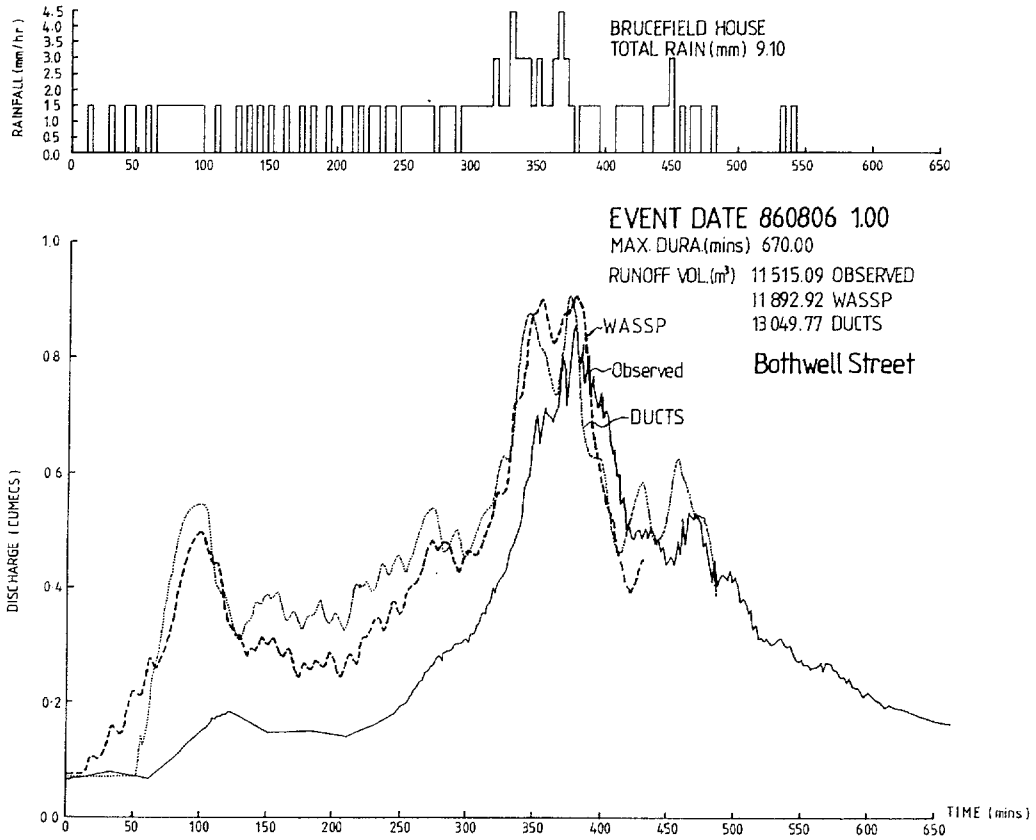


FIGURE 7.15 Comparison between Observed and WASSP and DUCTS Computed Flows at the outfall of Bothwell Street Chamber

FIGURE 7.16 PEAK DISCHARGES OF WASSP AGAINST OBSERVED VALUES

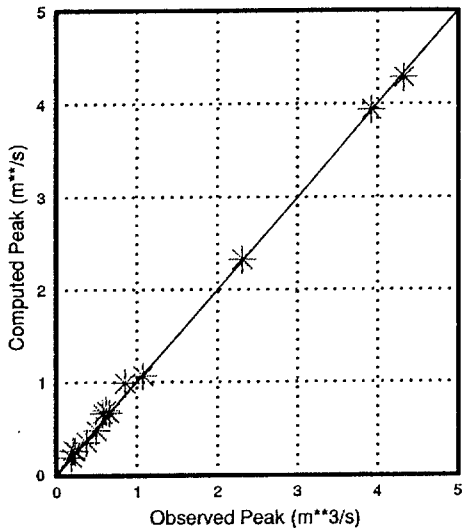
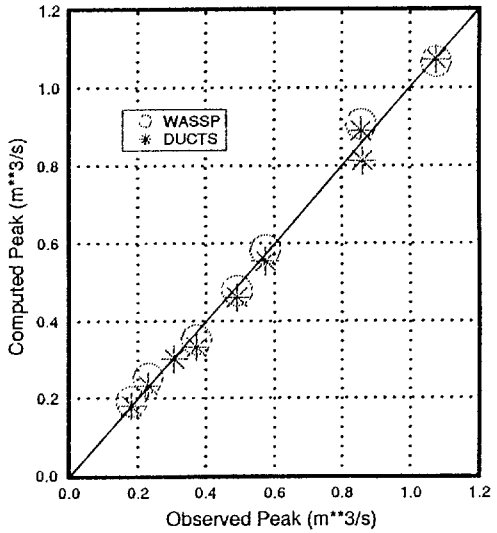


FIGURE 7.17 PEAK DISCHARGES OF DUCTS & WASSP AGAINST OBSERVED



CHAPTER 8
A PARALLEL PIPE MODEL

8.1 INTRODUCTION

A lumped pipe model was constructed for the Lyneburn system with the contributing areas simplified. The sewered sub-area model (SSA) was used to a great extent for model simplification. The major drawback of the single pipe model was that it only predicted the total outflow at the Bothwell Street overflow chamber without giving the amount of discharge, or percentage of flow in the two separate pipes. Without an enhanced model, prediction of the effects of rehabilitation of the system would be impossible. The amount of inflows to the treatment works and those entering the watercourse through the storm relief pipe during a storm required to be known. The commercial package, WASSP and the in-house simulation model, DUCTS were both used to simulate the combined pipe system and close fits were produced. The DUCTS model, therefore, was used as a basis upon which to develop the parallel pipe system. A major result, however, of the single pipe model was that the data used for the contributing areas could be considered to be reliable.

The Lyneburn parallel sewers originate at Halbeath Road, that is, where the Bellyeoman Sewer and Halbeath Sewer meet (Figure 5.4). Flows in these two pipes combine at the upstream end of an ovoid tunnel and the parallel pipes re-start downstream from this tunnel (Figure 5.9). The parallel pipe model as applied to Dunfermline, therefore, starts at this location and it is also the first cross-connection overflow for the model. The study outfalls for the parallel pipes were chosen to be at the overflow chamber at Bothwell Street, upstream of another major inflow from Towerburn Sewer.

The section of parallel pipes upstream of the tunnel continue to be represented by the equivalent combined pipe model which forms a single inflow to the parallel pipe system. Figure 5.9 shows the detail layout of the parallel pipes and the locations of cross-connection.

8.2 DUPPERS PARALLEL PIPE MODEL

The most widely used model in the UK currently is the simulation based on the Wallingford Storm Sewer Package, WASSP. Like many other models, WASSP uses a kinematic wave routing of flows (HRS 1981) and is only suitable for dendritic systems. The more recent model WALLRUS (HRS 1989), is better at dealing with the overland flow parameters but is still based on the same pipe routing procedures, although backwater effects are possible. Another model developed by HRS, the SPIDA interactive drainage analysis, uses the conceptual slot method (Osborne 1985) and can be applied to looped systems with flat or reverse gradients where free surface backwater effects are important. However, no computer model was available to simulate flows and levels in parallel pipes with cross-connections directly, without making simplifications such as the 'lumped' pipe model, and this led to the development of the parallel pipe model.

The Dunfermline Parallel Pipe Simulation Model (DUPPERS) is an enhancement, based on the in-house DUCTS model (Ashley & Jefferies 1984). In the development of DUPPERS, the major enhancement and modifications were the incorporation of a parallel pipe system and a flow routing procedure in the twin pipes and cross-connections. The computer subroutines which deal with the overland rainfall-runoff process remained unaltered. The following were the objectives and performance specifications for the enhanced model :

(i) both twin pipes were to be recognised as having the same hydraulic importance;
(ii) the model should be able to determine the straight through flows, overflow or reverse flow for the parallel pipes;
(iii) computation of side weirs overflow and reverse flow for the two types of cross-connections;
(iv) identification and formation of surcharged sub-systems in the parallel pipe system;
(v) level computation for both free surface and pressurised flow conditions;

(vi) flow and level hydrographs should both be obtainable for any node;

(vii) subsidiary inflows should be accommodated at any nodes along the parallel pipe system; and,

(viii) sub-models in the existing program should remain unchanged, particularly the sewered sub-area model.

Amongst all of the above, the major enhancement to the DUPPERS model was the incorporation of the parallel pipe and cross-connection component into the original DUCTS model. Side-weir overflows and reverse flow between the parallel pipes were the most important flow phenomena and hence required close investigation before the model could be constructed.

8.3 FLOW CONTINUITY

The presence of the storm relief pipe in the system has the effect of reducing the peak discharges and overloading of the foul pipe by overflowing via the side weirs at the cross-connections during storms. Prior to a storm, low flows pass through in the twin pipes without the occurrence of overflow. Flow then spills over the single- or double-sided weirs entering the storm relief through the bridging pipe as the flow increases. As the storm severity increases, the parallel pipes and cross-connections become surcharged. Flow in the storm relief pipe increases rapidly in the pressurised condition and level balance becomes possible. No overflow can occur during head balance in both the foul and storm relief manholes. Any further increase in flow and resultant head in the storm relief enables the reverse flow to be conveyed back to the foul. Captured data suggested that reverse flow in any cross-connection in practise rarely happened and only occurred during pressurised flow (refer to Chapter 6 for details).

The above flow phenomena can be shown diagrammatically in Figure 8.1. The flow pattern at a cross-connection can be represented by

the following flow paths :

(i)	Foul inflow	(Qfin)
(ii)	Storm relief inflow	(Qsin)
(iii)	Overflow	(Qover)
(iv)	Reverse flow	(Qrev)
(v)	Foul outflow	(Qfout)
(vi)	Storm relief outflow	(Qsout)

The relationships for the above flow paths in the parallel pipes and cross-connections can be represented by the general continuity Equations 8.1 and 8.2 as :

$$Qfout = Qfin - (Qover - Qrev) \quad (8.1)$$

$$Qsout = Qsin - (Qrev - Qover) \quad (8.2)$$

Due to the complex flow behaviour in a cross-connection, the above flow paths required to be clarifed in the enhanced parallel pipe model. The new model had to be able to identify the surcharged pipes in the parallel pipe system and to form surcharged sub-systems for different locations along the system with or without the presence of cross-connections (fuller descriptions have been presented in Chapter 9). All flow behaviours possible have been incorporated in the enhanced model.

8.3.1 STRAIGHT THROUGH FLOWS

In the normal dry weather flow and when overflow is not occurring, i.e. Qover = 0, the outflows in the parallel pipes at cross-connection are the same as the inflows. Equations 8.1 and 8.2 become :

$$Qfout = Qfin \quad (8.3)$$

$$Qsout = Qsin \quad (8.4)$$

Since reverse flow does not occur in the free surface flow, Qrev is not valid in the above equations.

Each pipe has its own conveyed inflow from the upstream pipe. However, inflow into the first storm relief pipe is the overflow in the bridging pipe from the foul at the upstream end of the system. The inflow to the storm relief pipe (Qsin) in the first cross-connection is not valid and Equation 8.2 becomes :

$$Qsout = Qover \tag{8.5}$$

For the case of no overflow in the first cross-connection, Equation 8.5 reduces to :

$$Qsout = 0 \tag{8.6}$$

Hence there is no outflow in the storm relief pipe at the first cross-connection.

8.3.2 OVERFLOW

Overflow commences whenever the flow depth in the foul manhole is higher than the crest of the side weir caused by increased inflow. Overflow can occur under both free surface and pressurised flow conditions as long as the driving head in the foul pipe is higher than that in the storm relief pipe. Once overflow becomes possible, Equation 8.1 and 8.2 become :

$$Qfout = Qfin - Qover \tag{8.7}$$

$$Qsout = Qsin + Qover \tag{8.8}$$

The above relationships are valid when reverse flow is not occurring. The overflow behaviour is shown in Figure 8.1.

8.3.3 HEAD BALANCE

Flows in the storm pipe increase at a much faster rate than in the foul during a storm because of the occurrence of overflow in cross-connections. The heads in both foul and storm manholes can eventually be balanced through increasing inflow in the storm

relief manhole. When there is a complete balance of heads, no flow can occur at a cross-connection inside the bridging pipe. The continuity relationship of flow paths represented by Equations 8.1 and 8.2 then becomes :

$$Qfout = Qfin \quad (8.9)$$

$$Qsout = Qsin \quad (8.10)$$

Conditions of true flow balance are likely to last only for a very short time due to the rapid changes and unsteady conditions which occur during surcharged flow in both foul and storm relief pipes. Any further increase of inflow in the storm relief manhole will cause the flow to be conveyed back to the foul pipe provided the head in the foul remains the same.

8.3.4 REVERSE FLOW

When there is overflow at a cross-connection, excess flow in the foul pipe is allowed to be discharged into the storm relief pipe. The proportion of flow is therefore much larger in the storm sewers than in the foul for heavy rainfalls. With further increase of flow in the storm relief pipe, reverse flow could occur back to the foul through the connecting pipe.

Close examination of the cross-connection layout together with the captured flow data (refer to Chapter 6) shows that reverse flow may start once the flow level is higher than the soffit level of the storm pipe in the lowest cross-connection.

Whenever reverse flow occurs, the flow behaviour alters and Equations 8.1 and 8.2 become :

For Qover = 0,

$$Qfout = Qfin + Qrev \quad (8.11)$$

$$Qsout = Qsin - Qrev \quad (8.12)$$

As seen from the above continuity relationships, reverse flow behaviour is in the opposite direction to the overflow at a cross-

connection, i.e. Qrev equals -Qover, but has been distinctly identified for clarity. The computed reverse flow as shown in Equation 8.11 is combined with the inflow to form the outflow in the foul sewer.

8.4 HYDRAULIC ASPECTS OF FLOW AT CROSS-CONNECTIONS

The primary function of the cross-connections is to allow flow to cross from the foul to the storm relief pipe to restrict the volume of through-flow to the treatment works during a storm. The hydraulic structures to be modelled at the cross-connections are the stormwater overflows in the foul manholes. An overflow setting is normally expressed as a multiple of the dry weather flow (Saul 1988). Recommendations on the selection of the setting is normally based upon Formula A which is given in the Technical Committee report on stormwater overflow (HMSO 1970) and in the Sewerage Rehabilitation Manual (WRC/WAA 1983).

There are many types of overflow structure, the main ones being :

(i) Leaping weir
(ii) Stilling pond
(iii) Siphon
(iv) Vortex
(v) Shaft
(vi) Side weir

The leaping weir was an old type of overflow and is not common nowadays. The amount of overflow depends upon the position of the leading edge of the overflow trough (Balmforth 1985[1]) and spill is normally over-estimated due to a lack of understanding of their hydraulic performance.

The stilling pond was developed by Sharpe and Kirkbride (1959) to provide good separation and retention of polluting solids (Frederick and Markland 1967). It has been widely used in new installations and was further tested and improved by Halliwell and

Saul (1980) and Balmforth (1982). A throttle pipe or orifice at the downstream end of the chamber is normally required to ensure proper hydraulic control and to fix the setting (Pratt & Balmforth 1986). Recommended chamber dimensions were also given by Balmforth (1985[1]). Fuller descriptions have been enclosed in the latest WRc report (Balmforth & Henderson 1988).

An earlier type of siphon fitted with an air-breaker pipe was designed to run completely full for most of its working range and is called Blackwater Siphon (Braine 1957). The siphon overflow utilises the difference in water levels between the overflow chamber and the storm outlet and is effective where space is limited or a limited head is available. Recently an air-regulated siphon has been developed and its performance has been investigated by various researchers (Ali and Pateman 1980, Balmforth et al 1982) using physical models. Problems in the operation of the blackwater siphon were solved by using the new siphon overflow. In particular the upstream water level can be regulated automatically over a wide range of discharges (Ervine 1976).

The concept of ulitising vortex motion in a circular chamber to separate settleable solids has been widely demonstrated (Smisson 1967, Field 1972 & 1974). Recent extensive investigations into their hydraulic performance was carried out (Balmforth et al 1984) by varying model chamber dimensions. The use of a spiral scumboard can also prevent floatables passing over the weir (Balmforth 1985[2]). Vortex overflows also give effective hydraulic control and good solids separation but require a drop in invert.

Similar in concept to the vortex overflow, the shaft overflow was also investigated thoroughly using a scaled model of chamber operation (Burrows and Ali 1982[1]) and of the retention of particles (Burrows et al 1984). The performance of the shaft overflow has also been compared and validated with the stilling pond overflow (Burrows and Ali 1982[2]).

The side weir is probably the most common type of overflow structure. The low weir was an early type which was used when the downstream sewer was the same size as upstream (HMSO 1970). The

amount of overflow and the storage required was examined at an early date (Braine 1947). Later, higher weirs were installed with downstream throttle controls and scumboards (Saul 1977). Performance of the high single and double weirs has been extensively investigated using laboratory tests and scale models (Balmforth 1978, Saul and Delo 1982). Hydraulic controls on the use of high side weir overflows were successfully achieved by carefully choosing the downstream throttle and chamber size (Balmforth and Sarginson 1978 & 1983, Saul and Delo 1981). Recently, design charts and graphical representation have been developed to aid the determination of chamber size, crest height and the number of side weirs (Delo and Saul 1989). Later research on the side weir overflow has been directed towards a better understanding of the hydraulic and solids separation by varying the weir height (Saul et al 1984, Saul 1985), and the effects of the first and secondary flush phenomena in the combined sewer overflow (Thornton and Saul 1986, Pearson et al 1986).

8.4.1 ANALYSIS OF SIDE WEIR DISCHARGE

Discharge over side weirs was widely approached using empirical formula (Coleman and Smith 1923) but this analysis failed to take into account velocity variations along the length of the weir. A simplified differential equation was later derived (De Marchi 1934, Ackers 1957) by ignoring the effects of channel slope and friction. In addition the specific energy was assumed to remain constant along the weir. The equation was solved by simple integration functions which were only applicable to rectangular channels. Later investigations carried out by Frazer (1957) gave the five different types of flow profile along a side weir. These are shown in Figure 8.2 and are described as :

Type I -- Approach flow is subcritical and as it is drawn down at the upstream end of the weir it reaches critical velocity. Supercritical velocities are present along the length of the weir and downstream of the weir is a hydraulic jump.

Type II -- Approach flow is subcritical with the weir crest set above the critical depth. As the flow passes along the weir the velocity is decreased and the depth is increased.

Type III -- Intermediate case between Types I and II. Upstream conditions being those in I and the downstream as those in II.

Type IV -- Supercritical flow is present throughout, with a falling water profile along the length of the weir.

Type V -- Intermediate case between Types IV and II with a hydraulic jump occurring along the length of the weir.

Some complete solutions to deal with above surface profiles were presented by various researchers (Chow 1959, El-Khashab and Smith 1976). The finite difference equation is solved using an iterative step-by-step method in which the rise in elevation of the water surface Δd, over a finite length Δx in the direction of flow is computed from the following equation :

$$\Delta d = \frac{\alpha Q_1 (V_1 + V_2)}{g (Q_1 + Q_1)} \Delta V \left[1 - \frac{\Delta Q}{2Q_1} \right] - Sf \, \Delta x \qquad (8.13)$$

The term ΔQ in the above equation can be solved by the transverse weir equation as following and it is computed for each timestep :

$$\Delta Q = (2/3) \, w \sqrt{(2g)} \, Cd \, H^{3/2} \qquad (8.14)$$

As demonstrated by Balmforth (1978), equation 8.13 can be rearranged for a channel with small gradient :

$$\frac{dd}{dx} = \left[So - Sf + \frac{vq}{gA} (2\beta - 1) \right] \Big/ \left[1 - \frac{\beta Q^2 b}{gA^3} \right] \qquad (8.15)$$

Equations 8.15 together with 8.14 are used to predict the discharge and the surface profile along the weir. The method is applicable to the non-rectangular sections.

8.4.2 OVERFLOW COMPUTATION AT CROSS CONNECTIONS

Discharge capacities over side weirs have long been investigated and different formulae were established (Engels 1917, Coleman and Smith 1923, Babbitt 1953). Most of these formulae are only applicable to particular models and generally give high discharges over the weirs. The formulae developed by Engels (1917) based on tests performed on large-scale models is :

$$Qw = 3.32\ L^{0.83}\ (d_2 - c)^{1.67} \qquad (8.16)$$

All units are in the foot-pound-second system.

Flow over side weirs in circular and non-uniform sections were later performed by various researchers (Collinge 1957, Allen 1957) based on the works of Nimmo (1928) and De Marchi (1934) and the five different types of flow profiles (Figure 8.2). These investigations gave similar results to Equation 8.16. More recently the transverse weir formula (Equation 8.14) has been widely used.

There are generally three types of overflow in the cross-connections in the Lyneburn system and they are shown diagrammatically in Figure 8.3. The formula used in the parallel pipe model was the same as used in WASSP model (HRS 1981) for determining flow over the weir and it can be shown as :

$$Qover = Cd\ L\omega\ \sqrt{g}\ H\omega^{3/2} \qquad (8.17)$$

Equation was developed based upon Equation 8.16 and later works (Coleman & Smith 1923, Balmforth 1978). The head above weir ($H\omega$) was calculated by subtracting the weir height from the water depth found by equations such as 8.13 or 8.15.

Equation 8.17 is only applicable to the free-surface overflow into the bridging pipe (case A in Figure 8.3), that is, Qover is less than the full-bore flow of the bridging pipe. When the connecting pipe is becoming pressurised (case B), the overflow is no longer

controlled by the weir, but by the head in the foul manhole. The flow crossing over to the storm pipe is hence determined by using the orifice equation in the form :

$$Qorif = Cd\ Ao\ \sqrt{(2gHo)} \qquad (8.18)$$

For the case C in Figure 8.3, in which both foul and storm relief pipe are surcharged, overflow is determined by the level difference in the two manholes (Bettess and Price 1978) and the equation is :

$$Qover = \sqrt{\left(\frac{Hdiff}{\beta e} \right)} \qquad (8.19)$$

where Hdiff = Hf - Hst and βe is the constant determined from :

$$\beta e = \frac{32\ fe\ Le}{\pi^2\ g\ d\epsilon^5} \qquad (8.20)$$

Equation 8.19 is also true for the determination of reverse flow in cross-connections for which the level difference is Hst - Hf.

Besides the above, a loss of head due to local turbulence at manholes, bends, junctions and also entry and exit ends of bridging pipes were required to be considered. As the discharge in the pipe approaches or exceeds its full bore capacity, head losses become more important (Ackers 1959, Archer et al 1978). The head losses in bridging pipes was calculated from the equation :

$$\text{Head loss} = \frac{k\ V^2}{2g} \qquad (8.21)$$

Some typical values for the constant k in Equation 8.21 can be found in WASSP (HRS 1981). A value of 0.5 was used in the parallel pipe model to account for the entry and exit losses in bridging pipes.

8.4.3 COEFFICIENT OF DISCHARGE

The accuracy of calculation of the flow over a side weir in Equation 8.14 largely depends on the value of the discharge coefficient (Cd). Balmforth and Sarginson (1977) have shown that the value of Cd is not affected by the fact that side weir flow has a longitudinal component of velocity and that it can be calculated from the Rehbock formula (1929) as :

$$Cd = \left[0.602 + 0.083 \frac{(y - p)}{p}\right] \left[1 + \frac{0.0012}{(y - p)}\right]^{3/2} \qquad (8.22)$$

Matthew and McKeogh (1982) simplified the above equation from comparison of experimental results with observed values. This simplified Rehbock equation was used in the enhanced parallel pipe model and Cd was computed as :

$$Cd = 0.602 + 0.083 \frac{Hav}{p} \qquad (8.23)$$

Equation 8.23 was used for the single-side weir in the parallel pipe model. For the double-side weirs, the constant term in Equation 8.23 was doubled due to the fact that draw-down was the same along the weir (Henderson 1987, Saul 1989) and hence :

$$Cd = 1.204 + 0.083 \frac{Hav}{p} \qquad (8.24)$$

The value of Cd was computed for every individual side weir in the cross-connections and updated for every time step. The value of Cd for the orifice overflow in equation 8.18, however, was taken as 0.59 for all connecting pipes as suggested by Balmforth (1985[2]).

The effects of bed slope and linear taper in the chamber width was small as shown by Saul (1988). However, the length of all overflow pipes in the foul manholes in the Lyneburn system was short and slope mild, hence no linear tapering was required.

8.5 FLOW DEPTH CALCULATION

In order to compute the overflow under free-surface condition (Equation 8.17), the flow depth in the cross-connection was required to be found using some suitable method. The model DUCTS on which the enhanced parallel pipe model was based did not compute flow depth under free-surface condition and there was no level hydrograph output after simulation. The depth computation was therefore considered to be important and became one of the major enhancements in DUPPERS.

It is recommended that the Colebrook-White equation should be used to determine the hydraulic behaviour of storm sewers (Barr 1981 & 1986, HRS 1981). It applies not only over the whole range of turbulent flow including smooth, transitional and rough turbulent, but also for any size and surface encountered in the Dunfermline system. The Colebrook-White equation was hence chosen for the depth computation in the enhanced model in the form :

$$V = -\sqrt{(32gRs)}\ \log_{10}\left[\frac{Ks}{14800\ R} + \frac{1.255\ \nu}{R\sqrt{(32gRs)}}\right] \qquad (8.25)$$

The flow depth computed in DUPPERS was by means of the relationship between proportional discharge and depth which is shown in the hydraulic chart (HRS 1983[1]) and in Figure 8.4. Firstly the proportional discharge (Qp) was calculated based on Equation 8.25 but in the form :

$$Qp = \left[1 + \frac{\log Rp}{\log(3.7\ D/ks)}\right] \cdot Ap \cdot Rp^{1/2} \qquad (8.26)$$

The terms Ap and Rp in Equation 8.26 are proportional area and proportional hydraulic radius respectively. They both can be

calculated using the following equations :

$$Ap = \frac{\theta - \sin\theta}{2\pi} \qquad (8.27)$$

$$Rp = 1 - \frac{\sin\theta}{\theta} \qquad (8.28)$$

The varible θ in Equation 8.27 and 8.28 is the angle sustained from the centre of pipe to the water surface and can be found for corresponding flow depth.

The proportional discharge (Qp) was computed in the program for the range of no flow depth to just pipe-full in steps of 1mm by Equations 8.26, 8.27 and 8.28. The procedure was then repeated for three relative roughness values (ϕ) of 100, 500 and 1000. The relative roughness ϕ which is the ratio of diameter to the sewer roughness i.e. D/Ks, is also computed for each pipe. The Ks values used for the Lyneburn parallel sewers were: 3.0mm for foul and 6.0mm for storm relief pipes.

To determine the flow depth under free surface condition, the proportional flow found by the routed and full-bore flows for a particular pipe was checked against the library array which the above Qp and Dp (proportional depth) relationship had stored, together with the relative roughness for that pipe. Once the corresponding Dp was found, the actual depth was determined by the following :

$$\text{Flow Depth} = Dp \cdot \text{pipe diameter (D)} \qquad (8.29)$$

A fuller description of the application of the above flow depth determination procedure to the enhanced model DUPPERS has been shown in Chapter 9 and in the computer listing (Appendix A(viii)).

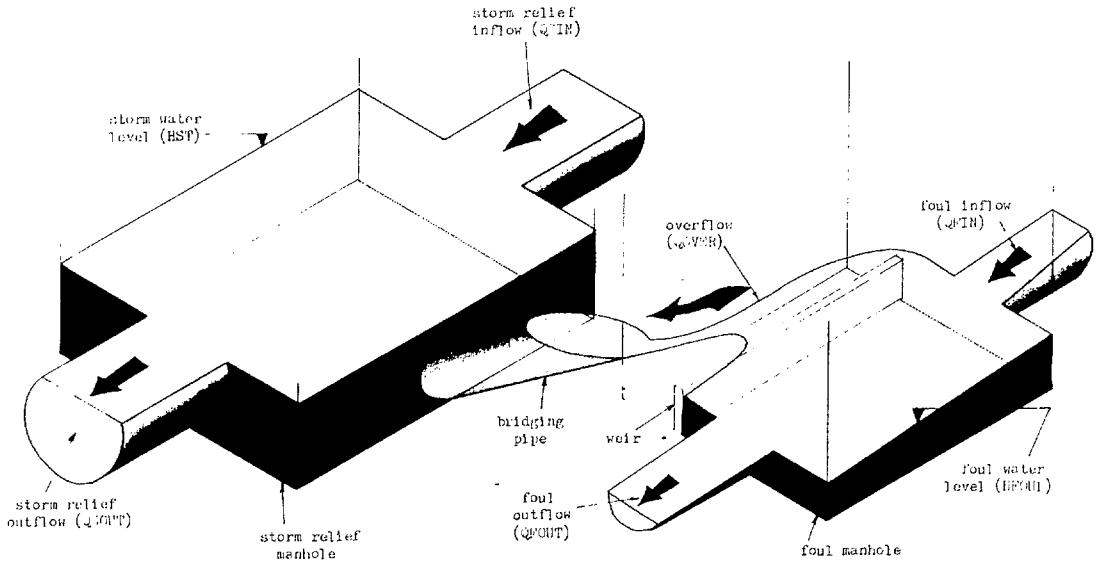
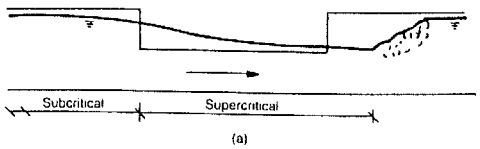
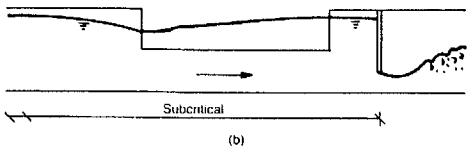


FIGURE 8.1 Diagrammatic Sketch of Overflow at Cross-Connection

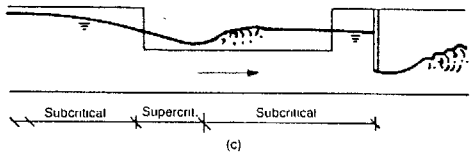
(Note : Reverse flow (QREV) is not shown in this case)



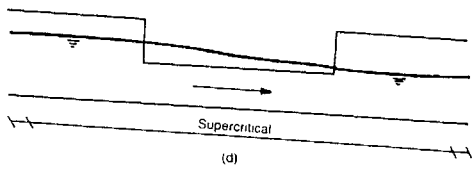
(a) Type I -- Mild Slope, Low Weir



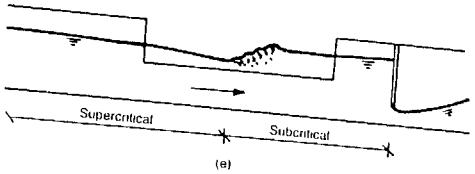
(b) Type II -- Mild Slope, High Weir, Downstream Throttle



(c) Type III -- Mild Slope, Low Weir, Downstream Throttle

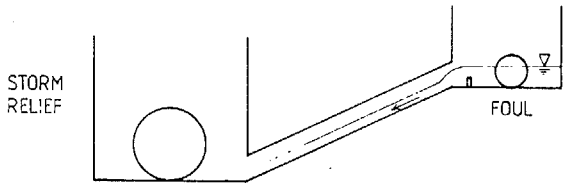


(d) Type IV -- Steep Slope, Low Weir

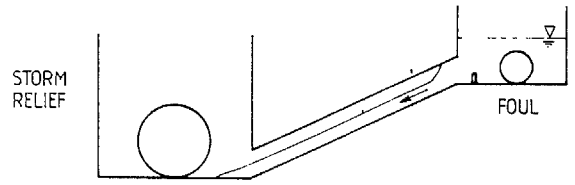


(e) Type V -- Steep Slope, Low Weir, Downstream Throttle

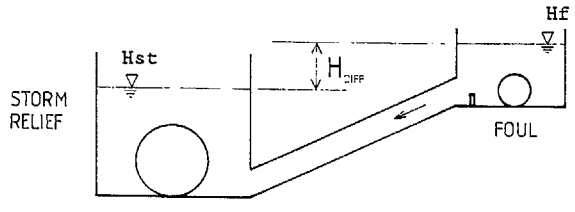
FIGURE 8.2 Classification Of Side-Weir Overflow



(a) Free surface overflow



(b) Pressurised overflow and free surface flow in storm



(c) Pressurised overflow due to head difference

FIGURE 8.3 Possible Overflow Behaviour in Typical Cross-connections

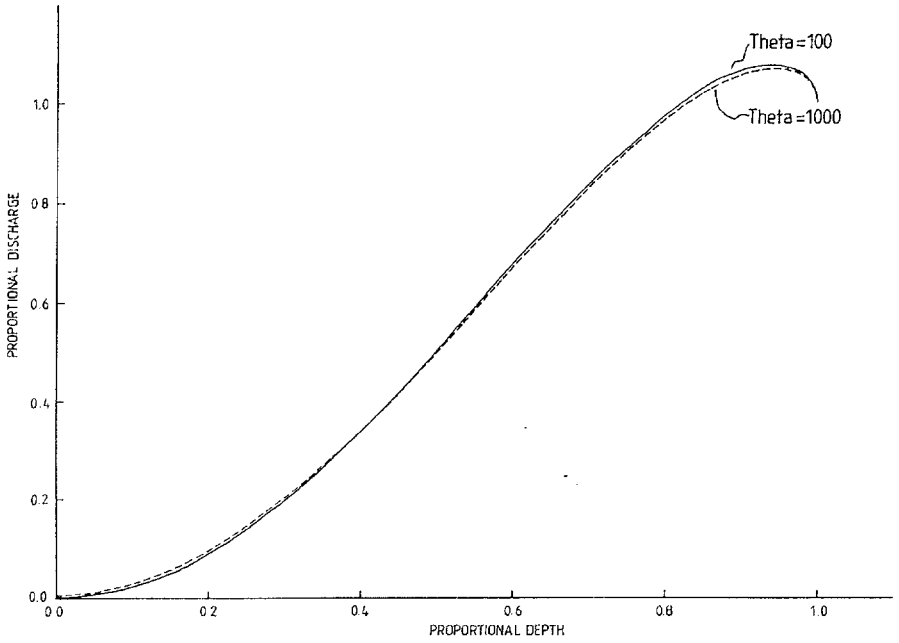


FIGURE 8.4 Relationship Between Proportional Discharge and Proportional Depth

(rough turbulence)

CHAPTER 9
DEVELOPMENT OF THE DUPPERS SIMULATION MODEL

9.1 INTRODUCTION

In the development of a computer simulation model for the Lyneburn parallel pipe system, there are difficulties in applying a standard package, centred around the duplication of a significant proportion of the major sewers with the presence of cross-connections between the twin pipes. Three alternative approaches appeared to be possible for the modelling of the study system :

(i) Adoption of the available algorithms for hydraulic structures in WASSP such as storm overflows or offline tanks.
(ii) Separation of the twin pipe system into two dendritic systems with some form of proportional contributing areas into each.
(iii) Combination into one single equivalent pipe model.

The overflow from offline storage tanks could not be used as a secondary pipe to give appropriate outflow at the final outfall making the first option unworkable. The second option would only be possible if there were no cross-connections between the twin pipes. The final option was a viable modelling technique but it could only produce combined discharges at a single outfall. The information obtained by this model could, however, be used to give a thorough understanding of the catchment and be used to verify the hydrological data (Chapter 7 gives details of the lumped-pipe model).

The in-house model DUCTS (a description of the model has been given in Chapter 4) was used as a basis for enhancement which could allow modelling of the parallel pipe system after the operation and performance of the lumped-pipe model was found to be satisfactory. The enhanced sewerage system simulation program which was developed

particularly for the Lyneburn system is named separately from DUCTS and hereafter is referred to by the acronym DUPPERS (Dunfermline Parallel Pipe Research Simulation).

9.2 ENHANCEMENT TO DUCTS

The simulation program DUCTS was written in FORTRAN-77 and at the start of the research, it ran on a DEC-20 mainframe computer. The entire software had to be modified after a cluster of VAX mainframe computers replaced the DEC-20 machine in 1987. The modified DUCTS is now in VAX-FORTRAN format and runs on the VAX/VMS V5 system (Digital Equipment Corp 1984).

Both the DEC-20 and VAX mainframe versions of DUCTS were formulated only for dendritic systems and in many ways are similar to the WASSP package. Two different types of data files are required as input to the program, these being the Sewer System Data (SSD) and Program Control Data (PCD) files. The SSD file consists of both overland information and below-ground pipe data which are represented by different numbers of columns for a system. Normally a row of data represents one pipe or a length of sewer, except for storage tanks which occupy two data lines. A fuller description on the input data has been included in a later section of this chapter. The PCD file contains mainly the rainfall hyetograph together with some global catchment information and the antecedent wetness condition.

The Program operates through a series of subroutines. The seven major subroutines located in the main program are as shown in Figure 9.1 and Table 9.1, and a further seven minor ones are contained within the major subroutines. The overall programme structure is detailed in Section 9.2.5. It can be seen from Figure 9.1 and Table 9.1 that most of the routines are for system data management and above-ground components. All the flow routing procedures take place in the main FLOW subroutine where output information is also produced.

Inflows from contributing areas into the trunk sewers are possible in two ways, both being represented by a single data line in the SSD file. The first is the single pipe data with the corresponding contributing area and other pipe details input for this pipe. The second is the utilisation of the Sewered Sub-Area model (SSA) with a sewered sub-catchment being represented by a single pipe (details and applications of SSA have been described in Chapter 4). In DUPPERS, SSA is recognised by using the ancillary index (NANI) number 3. A fuller description on the NANI and branch numbers is detailed in Section 9.4.1.

Major enhancements in DUPPERS include the recognition of a parallel pipe system and the flow routing in these twin pipes and cross-connections. The parallel pipe data input starts in the first subroutine ENDORD and other minor modifications were also necessary in the following subroutines. Modifications for the flow routing process in the parallel pipes are focussed on the below-ground hydraulic flow sub-model and hence all the above-ground hydrological components remained unaltered. The additions and modifications of DUPPERS which are based on DUCTS are shown in Figure 9.1 together with brief descriptions of the functions of each subroutine. Table 9.1 gives the details of the enhancements.

9.2.1 COMPUTING STRATEGY

Although the primary aim of the research was to produce a parallel pipe model, in order to allow the dendritic system also to be simulated by DUPPERS, as much as possible of the original version of DUCTS was left untouched in the enhanced model. In view of the large size of the existing DUCTS program, enhancement of the new model was by adding procedures to deal with parallel pipe systems in existing subroutines rather than by creating new subroutines. Furthermore, a large number of additional arrays had been created in the new model.

The main enhancements of the new model were its use on parallel pipe systems and the flow routing in the twin pipes. The method of reading in sewer system data for parallel pipe systems in DUPPERS is similar to the original dendritic pipe model. Each line in the

SSD file represents a single pipe data or SSA information for a subsequent sub-catchment. A complete cross-connection overflow including the bridging pipe requires two data lines due to the increased input data and their formats.

The numbering system in DUPPERS is again similar to the dendritic systems in DUCTS, that is, number 1.0 is assigned for the main branch of a system with the longest sewer length. However, two unique numbers (500 and 600) were chosen for the foul and storm relief pipes in the parallel sewer system and these are detailed in the next section. In a similar manner to dendritic systems, DUPPERS utilises the end-order (ENDORD) numbering procedure in which a manhole number is assigned for each pipe at the upstream end by the program for the parallel pipe system. The determination of the ENDORD numbers is a vital computational procedure because it stores all system information including inflows and outflows for a particular node and pipe.

The computational procedure for determining the ENDORD number in DUPPERS has been organised to accommodate each of the parallel pipes. Only after the ENDORD numbers have been correctly assigned, are the flows in the twin pipes stored separately. In the downstream manhole number computation, both for the foul and storm relief sewers, is treated as two identical but separate main branches with branch number set to 1.0 but reset back to 500 and 600 afterwards. Any major inflows can also be input to the parallel pipe system as in the normal data input procedure. Figure 9.2 shows a typical ENDORD manhole number sequence for (a) dendritic system and (b) parallel pipe system. The computed downstream manhole number is stored in the array MDOWN in the first subroutine ENDORD. The computer listing for the determination of MDOWN is shown in Appendix A(i). It can be seen from the listing that branch and pipe numbers are represented by IB3 and IP3 respectively. Other branch numbers apart from the parallel pipes are dealt with in statement 50 whilst parallel branches are in statement 48. The procedure for computing the MDOWN numbers is fulfilled by the DO loop 170.

In the flow routing procedure in DUCTS, two different timesteps are used for the free-surface and pressurised flow conditions and these

are 10 seconds and 1 second respectively. The choice of 1 second timestep in the surcharged condition is primarily to deal with the rapid change in flow and level in very short time intervals. The same timesteps are used in DUPPERS. In the free flow condition, flow routing computations are first carried out for the foul pipe till the next downstream cross-connection overflow is reached. The routing procedure is then performed for the storm relief sewer and again halts at the next overflow. This procedure is repeated for the entire parallel pipe system within the particular timestep. In the surcharged flow mode, however, the determination of the extent of a surcharged sub-system and the flow solution are all performed simultaneously in the shorter timesteps for the whole sub-system.

9.2.2 PARALLEL PIPES

A pipe is represented by branch and pipe numbers for both dendritic and parallel pipe systems in the SSD files.

The twin pipes in any parallel pipe system have equal importance in flow and hence the pair of pipes have to be input together. The numbering system for the parallel pipes in DUPPERS assigns each of the parallel pipes a unique number. The branch number assigned for the foul sewer is 500 whilst 600 is used for the storm relief sewer. The numbers 500.1 and 600.1 hence indicate the first pipes of the foul and of the storm relief respectively at the beginning of the parallel pipe system. The twin pipes are set out in discrete sections in the SSD file. Figure 9.3 shows a typical section of parallel pipes containing a cross-connection overflow and a group of foul and storm relief pipes upstream from the next overflow.

Subsidiary branches or inflows from SSAs are allowed to join at any node on either branch of the parallel pipes. This allows flow from any subsidiary parallel pipes to enter the main twin sewers. A pair of dummy parallel pipes are required at the downstream end of the system to allow the simulated outputs to be stored and printed out.

9.2.3 CROSS-CONNECTION

In simulating the flow behaviours in a cross-connection overflow (CCO), the level in the overflow governs both the direction and amount of discharges, particularly when computing overflow and reverse flow. To allow the above flow regimes to be determined in a cross-connection overflow (CCO), the physical dimensions and all relevant levels such as inverts of the twin pipes and crest of weir require to be included in the input data. All information for each single CCO in the system are input in two successive data lines in the SSD file, one for the overflow and the other for the bridging pipe. Data in these two lines are separated by DUPPERS, with the overflow pipes under the foul sewers and bridging pipes grouped together with storm relief sewers at the appropriate locations. As for the parallel pipes, separate unique numbers are employed for the overflow pipe and bridging pipe of the CCO. The numbers 700 and 800 are used for the overflow and the bridging pipe respectively with a decimal number indicating the location of the CCO, for example, 700.1 and 800.1 indicate the first CCO in the system. Figure 9.3 shows the standard numbering system for a CCO.

It can be seen from Figure 9.3 that two imaginary manholes are set up by the program DUPPERS during simulation. The purpose of these manholes is to store the inflows and outflows upstream and downstream of the overflow pipe, no storage of flows during surcharge being allowed. However, storage in the surcharged condition is catered for in the 700 manhole itself and this is based on the manhole plan area and the surface level.

Although inflow from the subsidiary branches and SSAs are permissible, they cannot be input into a CCO directly due to the problem of computing the downstream manhole number. However, this is overcome by inserting a very short length of pipe just upstream of the CCO. The combined inflows into the upstream end of the CCO is then checked for overflow and throughflow.

9.2.4 SYSTEM DATA INPUT AND OUTPUT

DUPPERS distinguishes between dendritic and parallel pipe systems by the branch numbers assigned for the twin pipes. The branch number 1 is recognised by the model as the main branch in dendritic system. In the presence of branch numbers 500 and 600, DUPPERS then follows the enhanced procedures for parallel pipe system. Due to the unique numbering system, both dendritic and parallel pipe systems can be mixed in a simulation network and still be recognised by DUPPERS. Any other subsidiary branches or SSAs in the parallel pipe system can be input in the normal procedure.

The required input data for parallel pipes 500 (foul), 600 (storm relief) and 800 (bridging pipe) are the same as for a normal pipe in a dendritic system. For the CCO, however, input data are slightly different and the required data include the weir type and dimensions, levels and optional Cd values. A fuller description on the input data types and corresponding formats for parallel pipes and CCO are included in Section 9.4.1.

After a complete simulation, DUCTS prints all output into two separate data files. One of these is the data checking file containing all the system information and hydraulic details of subsidiary branches, SSAs and normal flow conveying pipes. The other has the computed simulation results including the smoothed rainfall hyetograph and all discharge hydrographs at one minute intervals. Appendix A(ii) is for the output of the CCO data after the initial data read-in procedure.

DUPPERS gives simulation output to two separate data files DUPCHK and DUPOUT. Output formats largely remained unaltered but have been modified to give additional information in the output data files. Although the data for overflow and bridging pipes are input together, the bridging pipe is separated from the overflow. Output data from this pipe is located before the downstream storm relief pipe at a particular CCO, and the entire sewer system after the reorganisation is printed in the check file DUPCHK. In addition to sewer system details, information on the cross-connections are also reproduced in DUPCHK for input data checking. All input data and

the downstream manhole numbers for the overflow and straight-through flow are also given in the checking file. Formerly, DUCTS was only able to output discharge hydrographs to the output data file for those nodes designated by DUCTS. The enhanced DUPPERS is able to output level hydrographs in conjunction with the discharge hydrographs for assigned manholes into the file DUPOUT.

Separate computer software GPLOT was also developed by the author specifically to read the discharge and level hydrographs and to produce graphical plots. Observed hydrographs can also be plotted to allow visual comparisons. Furthermore, information such as runoff volume (RUNVOL) for each discharge hydrograph is also computed and given on the plotted graph. The software GPLOT is enclosed in Appendix B.

9.2.5 PARAMETER STORAGE

The early version of DUCTS was developed to simulate small systems and all parameter sizes and the number of arrays for data storage were rather small. In order to test the performance and to detect computer programming errors, real catchments in Dunfermline with about 30 pipes were used for simulation (Angus 1985).

After model amendment, the new version of DUCTS was found to be working satisfactorily but lacked of comparison with observed data in order to improve the performance and verify catchment models. Any system having 300 pipes or less could be simulated by the new version of DUCTS, after the size of arrays was increased to 300. In the system simplification process (Chapter 7), the arrays for storing observed event data were further enlarged to accommodate storms of up to 8 hours duration i.e. 480 timesteps.

In the beginning of the enhancement procedure, it was decided that DUPPERS required to have the array sizes further increased in order to simulate the entire Lyneburn sewer system without reducing the actual number of pipes. After a review of the full catchment and system size and the captured event durations, the array size for storing system information was increased to 450 and initially only 5 CCOs were allowed. The size of the observed rainfall hyetograph

array able to be accommodated was also increased to 730 so that storms with 12 hours duration could also be used for simulation. However, with all these changes on the DEC-20 version, coupled with additional arrays in DUPPERS for the parallel pipes, the program failed frequently to give correct outputs due to conversion errors in the rounding up of numbers and to the mis-storing of information, especially in the case of simulations of the most severe events.

With the advantage of having virtually unlimited computer memory capacity in the VAX mainframe, DUPPERS was further upgraded in the array size and number of parameters for storage. At present, any system with less than 450 nodes or pipes can be simulated by DUPPERS. The total simulation time has also been increased to 1560 minutes in order to model those events with long rainfall durations.

A total of 170 arrays of different sizes are used in the VAX version of DUPPERS. Some of these are two-dimensional arrays for storing rainfall data, depth and discharge hydrographs, and also the overland catchment runoff. Out of the 170 arrays, 55 are for the sewer system data storage; 34 for pipe routing constants and SSA variables; 21 for above ground parameters; 12 for the output storage; and the others are for ancillary hydraulic structures such as storage tanks and storm overflows. All the arrays are linked by using COMMON blocks at the beginning of each subroutine. A typical computer listing for the COMMON blocks with the arrays is shown in Appendix A(x).

DUPPERS consists of a total of 7 major subroutines and they are called up in the appropriate sequence. Figure 9.4 is the tree-diagram showing all major and minor subroutines. As seen from the figure, the seven subroutines are named accordingly, with 3 for the system read-in and constants computation, 2 for the rainfall profiles and overland runoff routing, 1 for the below-ground sewer constants calculation and the last for the flow routing in pipes for both free-surface and pressurised condition. Another 7 minor subroutines are called within the main routines. Subroutine ERROR is to identify all recognisable errors and output to the files DUPCHK and DUPOUT.

9.3 COMPUTING PROCEDURE FOR FLOWS AT CROSS-CONNECTIONS

The potential flow behaviour during storms at cross-connections is complex and checking routines are set up in order to determine the appropriate flow regime which is to be modelled by the flow computation procedures relevant to that regime. Figure 9.5 shows the relevant features of the cross-connection and the arrays set up to store those important locations for each cross-connection. The usage of these arrays and the call-up procedure in the computer program have been shown in Appendix [A(iii)1].

The computational procedure follows the possible flow paths as described in the logic diagram in Figure 9.6. For flow entering the foul manhole at a cross-connection, the level is first computed (Appendix [A(iii)2]) and compared with some or all of the arrays shown in Figure 9.5, in order to determine the possibility of overflow. For a flow depth higher than D3, overflow is checked to ascertain whether it is free-surface or surcharged. Once the type of overflow has been determined (Appendices [A(iii)2] and A(iv)) the flow level in the foul is compared with the head in the storm relief due to the corresponding inflow, in order to check for head balance or reverse flow (Appendices A(v) and A(vi) respectively). Since the relevant hydraulic solution is different for each flow regime, the required computation is considered separately.

Before developing the CCO modelling procedure in DUPPERS, all possible flow behaviour in the five cross-connections were studied in order to simplify the enhancement works and to cover all flow conditions. Three possible flow regimes were identified based on observed data interpretation at downstream cross-connection (Chapter 6) : these were overflows, head balance and reverse flow. These flow behaviour were then classified into two groups according to the flow depth in both the manholes. The first was overflow under either free-surface or pressurised condition and the other was flow balance and reverse flow. It was also noticed that the second group occurred only under complete surcharge across the cross-connection. Figure 9.7(a) shows all possible overflows in CCO including free-surface and surcharged whilst 9.7(b) shows the head balance and reverse flow. Table 9.2 lists the different types

of overflow and the condition statements corresponding to Figure 9.7(a). Similarly, Table 9.3 gives the computer condition statements for head balance and reverse flow behaviours as shown in Figure 9.7(b).

In the following sections, continual reference is made to Appendix A which has fourteen subsections. These are referred to as, for example, [A(iv)1].

9.3.1 FREE SURFACE OVERFLOW

As soon as the depth of flow in the foul manhole (YCO) is above the weir crest level (D3) (see Figure 9.5), overflow starts entering the cross-connection. YCO is considered to be free surface when it is less than the soffit level of the bridging pipe (CNSL) and also the total overflow is less than the full-bore capacity of the bridging pipe [A(iii)2].

Free surface overflow is computed using Equation 8.17 with coefficients of discharge from Equations 8.22 and 8.23 for the single- and double-sided weirs respectively. The algorithms for the determination of coefficients of discharge (CD1) and the amount of overflow (QCRO) are shown at 759 in [A(iii)2]. CD1 is computed according to the overflow weir type i.e. single- or double-sided weir. The array IWT is used to store the weir type for all CCOs in the system.

YCO governs both the overflow and the corresponding outflow for a particular timestep and it is therefore important to compute both simultaneously [A(iii)2]. There are two possible conditions for outflow through the downstream foul pipe, these being either free-surface or surcharged flow. The free-surface outflow is computed as the difference of the inflow and the overflow (788 in [A(iii)2]). The level in the outgoing pipe is determined by the level computation routine which includes the relationship for proportional discharges and depths stored in the two-dimensional array DPROP [A(viii)1]. Fuller description of the level computation is shown in Section 9.4.2. When the outgoing foul pipe

is surcharged, the outflow is determined by the orifice equation i.e. Equation 8.18 and formation of surcharge sub-system begins with the outgoing pipe.

An iteration procedure, which is shown by 781 and loop 759 in [A(iii)2], is used to determine the overflow and outflow with respect to the known inflow using the continuity relationship, i.e. Equation 8.7. This procedure starts with the level computed in the last timestep as the first approximation until the sum of overflow and outflow is within ±1% (781 in [A(iii)2]) of the inflow, in order to balance the demand for accuracy of the solution and with the cost of the computer run-time.

The overflow computed in this iteration is stored in the variable QCRO and summed with the inflow in the storm relief pipe (780 in [A(iii)2]). A level is then found based on these combined inflows in the storm relief in order to determine the possibility of reverse flow (see Section 9.3.4) and surcharged outflow.

9.3.2 PRESSURISED OVERFLOW

The procedure for the determination of pressurised overflow in a bridging pipe (conditions (iv) and (v) in Figure 9.7(a)) and the computation are shown partially in Appendices A(iii) and A(iv) for free-surface and surcharged flow at storm relief pipe respectively. The different types of surcharged overflow and their required conditions are shown in Figure 9.7(a) and Table 9.2.

Once the overflow condition is met, i.e. YCO > D3 (782 in [A(iii)2]), the overflow type is determined by comparing the reduced level of flow depth in the foul (YRLF) and the soffit level of the bridging pipe (CNSL). The overflow is considered to be pressurised if the flow depth is above the soffit level i.e. YRLF > CNSL or depth of flow exceeds the diameter of the bridging pipe [A(iv)].

There are two possible overflow behaviour once the bridging pipe is surcharged. Both are determined by the reduced level of flow

(YRLS) which is in turn governed by water depth (YSW) in the storm relief manhole and the outlet soffit level of the storm relief pipe (CRSL) and they can be represented by the following conditions :

(i) CNSL $\leq$ YRLF and CRSL > YRLS
(ii) CNSL $\leq$ YRLF and CRSL $\leq$ YRLS

In the first condition, free surface exists at the downstream end of the bridging pipe. This is in effect overflow type (iv) in Figure 9.7(a). Overflow under this condition is computed using an orifice equation as shown by Equation 8.18 and in [A(iii)2] and [A(iv)3]. The procedure at 759 in [A(iii)2] deals with the case when the foul outlet is under free surface flow.

When the bridging pipe is completely surcharged as shown in type (v) in Figure 9.6(a), overflow is calculated according to the difference in heads and Equation 8.19 is used for the overflow computation (40 in [A(iv)3]).

The pressurised overflow computed for the above flow conditions in CCO is stored in the array QCRO in a similar manner as in the free-surface overflow. The QCRO is again combined with the upstream inflow in the storm relief manhole at a CCO.

9.3.3 FLOW BALANCE

Figure 9.7(b) and Table 9.3 show the conditions for the occurrence of the head balance. The computer listing for this flow regime in a CCO is included in Appendix A(v).

During surcharge condition, storm flows can be stored in manholes along the parallel pipes. In CCOs, additional storage is possible in the bridging pipes. During severe storms, large amounts of flow cross over from foul to storm pipes causing flow level in the storm relief manhole of CCO to rise rapidly. The heads in both the foul and storm manholes become finally the same and hence there is flow balance in the CCO. Table 9.3 gives the necessary conditions for the flow balance behaviour to occur in a cross-connection.

The checking of flow balance in DUPPERS is by comparing the levels of flow in both the foul and storm relief manholes [A(v)1]. The computed reduced level of foul flow (YRLF) based on the foul inflow is checked with the reduced level in the storm relief side (YRLS) [A(v)1]. When these levels are equal, i.e. YRLF = YRLS, the heads are balanced on both sides and no cross flow in the bridging pipe occurs [A(v)2].

When the heads are in balance, no overflow is computed in the flow balance procedure. Subsequently, both the inflows entering the CCO equal the outflows of the parallel pipes. Equations 8.9 and 8.10 show the continuity relationship for the flow balance behaviour.

9.3.4 REVERSE FLOW

The second illustration in Figure 9.7(b) and Table 9.3 shows the conditions for the occurrence of reverse flow in CCOs. The computational procedure for this flow regime is included in Appendix A(vi).

For flow to travel to the foul sewer from the storm relief, the head in the storm manhole has to be higher than that in the foul, i.e. YRLS > YRLF (Appendix [A(vi)1]). Once this condition is satisfied in DUPPERS at a particular timestep, the reverse flow (QREV) will then be computed based on the difference of the heads using Equation 8.19 (51 in Appendix [A(vi)2]). After reverse flow has been computed, continuity of the flows in the twin pipes will be maintained, i.e. Equations 8.11 and 8.12 [A(vi)2].

The computed value of QREV is stored for summing with the flow at the inlet to form combined inflow in the foul pipe, i.e. statement QRMF = QRMF + QREV in [A(vi)2]. Stored QREV can also be printed out in the output file DUPOUT.

9.4 MANAGEMENT OF DUPPERS PARALLEL PIPE MODEL

The major enhancements of DUPPERS have been discussed and detailed in the beginning of this Chapter focusing on the recognition of parallel pipes and cross-connections, and the different flow regimes in the CCOs. This section then gives a further insight into other modifications which have importance in the DUPPERS model, including :

(i) Required data for parallel pipes and CCOs,
(ii) Level computation for free surface flows,
(iii) Flow routing procedure in parallel pipes.

The above inclusions together with those detailed in the previous sections of this Chapter form the main computational enhancements in DUPPERS. However, the model would not function at all without the basic programming framework, i.e. original DUCTS model. To ensure that DUPPERS was working satisfactorily before being put in use, numerous minor changes had been performed and these are too general and widespread to be included here.

9.4.1 REQUIRED INPUT DATA

The recognition of a parallel pipe system in DUPPERS occurs by the use of different unique numbers for the twin pipes and CCOs. The traditional branch number 1 used for the main sewer in dendritic systems remains in use. This is important as it allows dendritic systems to be simulated by the enhanced model. Secondary branches and inflows from SSAs can be represented by the numbers from 2.0 to 499.0, which is considered to be sufficient even for very large systems. Numbers 500 and 600 are assigned to the foul and storm relief parallel pipes. Numbers in between the two remain for potential use in modelling other uncommon systems such as double looped or triple pipes. In a similar way, 700 and 800 signify cross-connections and overflows respectively. The numbers assigned for sewers and CCOs have been summarised in Table 9.4 together with the ancillary index (NANI) allocated for the different hydraulic structures in the program.

The data read-in procedure in DUPPERS are included in subroutines ENDORD and DATIN, [A(vii)1] and [A(vii)2] respectively. Subroutine ENDORD only reads in the branch, pipe and NANI number in order to compute the MDOWN number (Section 9.2.1). The complete data read-in procedure is in Subroutine DATIN. A number sorting procedure according to the branch number is employed to distinguish the parallel pipes, overflows and bridging pipes and is shown after line 12 in [A(vii)1].

The required data for parallel pipes and CCOs are summarised in Table 9.5. It can be seen from the table that the data required for the parallel pipes are in fact the same as for other normal pipes including the sizes and levels of sewers. However, some specific data are needed for CCOs and bridging pipes and their input formats are different from the pipe data. The data for overflow and bridging pipe are shown in Table 9.6 together with the input formats and descriptions for each. An example of a sewer system data (SSD) file is included in Appendix A(xi) with the presence of CCOs.

The order of locating the parallel pipes in the SSD is the same as for a dendritic system but it is important that discrete sections have their correct format, i.e. a section of parallel pipes containing a CCO. The order of inputting the CCO is to put 700 cross-connection first, followed by the bridging pipe 800 data. These two data lines will be separated by DUPPERS once the data read-in procedure has been completed and the order of parallel pipes is reconstructed by the program by placing the 700 in the foul side whilst 800 is located with the storm relief. This separation of CCO data lines allows not only the downstream manhole numbers to be computed by the original routines as in DUCTS, but it also accommodates all possible sub-systems formed during surcharge.

9.4.2 LEVEL COMPUTATION

The general relationship between the proportional discharge and depth is computed by DUPPERS in the subroutine CONST so that flow depth for partially full flow can be computed after the routing

procedure at the downstream end of each pipe. The graphical relationship of proportional discharge to proportional depth has been shown in Figure 8.4 and full description on the derivation of the relationship is presented in Chapter 8. This proportional discharge to depth relationship is stored in the two-dimensional array DPROP for three relative pipe roughnesses i.e. D/Ks values of 100, 500 and 1000 (loop 800 in [A(viii)1]). The computer listing for setting up the array DPROP is also shown in [A(viii)1].

Partially full flow is computed by the Colebrook-White equation which has been shown as Equation 8.24. The proportional discharge, however, is found using Equation 8.25. The computational procedure for the determination of proportional discharge i.e. ratio of partially full flow to full-bore flow is by using two return loops in the program : the first to compute the flow for each 1 millimetre of depth (loop 850 in [A(viii)1]) and the second to repeat for different D/Ks roughness values as shown above (loop 800 in [A(viii)1]). The computational procedure for determining the proportional area (AP) and proportional hydraulic radius of pipe (ROP) has also been included in loop 850 in [A(viii)1]. The computed proportional discharge is then stored by the variable QPROP which in turn stored in the two-dimensional array DPROP in subroutine CONST (line 419 in [A(viii)1]). Subsequently the relative roughness value is calculated for each pipe in the system based on the diameter (D) and the corresponding pipe roughness value (Ks) and stored in array NDKS (loop 880 in [A(viii)1]).

The double array DPROP is subsequently recalled in subroutine FLOW [A(viii)2] after the flow has been routed through a pipe. Firstly the most appropriate D/Ks value is chosen from DPROP based on NDKS for the pipe (the term NDKS(J) in [A(viii)2]). The proportional discharge which is the ratio of the routed flow to the full bore flow is found (the term RAQ in [A(viii)2]). The proportional depth figure is then searched and determined from DPROP and the corresponding flow depth is thus obtained (YNN in [A(viii)2]). The computed level is stored in array YN for locations where a level hydrograph is required.

9.4.3 FLOW ROUTING IN PARALLEL PIPES

The flow routing procedure for the free surface condition is identical to those in DUCTS and performed on each pipe individually. In the parallel pipe system, routing is performed on the foul sewers first between CCOs and subsequently on the storm relief pipes. Flows in the parallel pipes just before the CCO are compared in order to determine the correct flow paths which are based on the levels in both manholes. The procedure is repeated for the next discrete section of parallel pipes.

The formation of surcharge sub-systems and the solution for the pressurised flow condition in DUPPERS are again based on those in DUCTS but required modification due to the presence of the CCOs between the twin pipes. The enhanced procedure for pressurised flow routing also requires to cover all possible surcharged sub-systems within the parallel pipes with or without the CCOs involved. Figure 9.8 shows diagrammatically such possible sub-systems and the computer listing for the pressurised flow routing procedure is shown in Appendix A(ix).

Due to the rapid change in levels during surcharged conditions, the timestep for the routing procedure is reduced to 1 second intervals (loop 250 in [A(ix)2]). For the case when surcharge occurs only along the twin pipes but not in the cross-connections i.e. condition (i) in Figure 9.8, two separate surcharge sub-systems are formed. However, for those situations with the bridging pipes surcharged i.e. conditions (ii), (iii) and (iv), the entire sub-system could include either part or even all of the foul, storm relief and bridging pipes and hence solutions are required to be obtained for the whole surcharged group.

To detect a surcharged pipe, the heads at both upstream and down-stream ends are compared based on the inflow to that pipe. When the downstream head is above the upstream one, the pipe is said to be surcharged and this is shown in the inequality in [A(ix)]. Alternatively, the level which is based upon the inflow to the upstream end is checked to see if it exceeds the invert level of the pipe. If the level is found to be higher than the invert

level, the pipe is then termed as surcharged. Under free-surface conditions, the upstream head is reset and is always equal to the invert level after the free surface flow routing procedure. When the inflow causing this head goes above the invert level during a particular time step, the pipe is therefore said to be surcharged (YY in [A(ix)1]). Once a surcharged pipe has been found, the checking procedure continues to the next downstream pipe. The procedure is repeated until inflow is less than the full-bore flow of the downstream pipe, or the head in the downstream manhole is less than the upstream head [A(ix)2] and hence the extent of a surcharged sub-system is found.

Structure matrices are then set up for the surcharged sub-system so that discharges and levels can be solved simultaneously for the entire sub-system (loop 90 in [A(ix)2]). Discharges and levels under steady state condition are firstly computed for the surcharged sub-system. The levels in the sub-system are then compared with the steady state levels. When the difference is in excess of 5% of the steady state values (loop 200 in [A(ix)2]), Runge-Kutta algorithm (Fox 1962) is used to calculate the depth at the 1 second time step for the sub-system (line 150 in [A(ix)2]).

Once the levels are computed for the sub-system which includes the parallel pipes and CCOs, the overflow, flow balance and reverse flow are computed in similar procedures as detailed in Section 9.3 in the surcharged routine [A(ix)2]. Hence discharges for the surcharged sub-system including the twin pipes and cross-connections are computed.

9.5 OPERATION OF DUPPERS

The simulation model DUPPERS requires two input data files one being the sewer system data (SSD) file and the other containing the rainfall hyetograph and control data (PCD). Sample input SSD and PCD data files are included in Appendices A(xi) and A(xii)

respectively. The SSD file is the Lyneburn parallel pipe system with all five cross-connections. A synthetic rainfall data of 2.0 mm/hr intensity continuously for 60 minutes is included in the PCD file.

The user can interactively define a standard percentage runoff (SPR) value and the extended simulation duration which can be up to an additional 100% of the rainfall duration. This second option allows a simulation of flows which are still left in the system after a heavy event.

The time taken for each simulation depends largely on the size of the system and the required simulation timesteps. Although there is a greater number of arrays in DUPPERS and more computation involved for a parallel pipe system, the time required for each complete simulation for a large system such as the full Lyneburn parallel pipe system with 8 hours rainfall duration is still less than with DUCTS using the DEC-20 mainframe computer due to the more powerful memory capacity in the VAX computer.

Simulation results are given in two separate output data files. The check data file (DUPCHK) lists the input data for the entire system, once again for checking together with constants computed for each pipe and SSA. The smoothed rainfall hyetograph and the hydrographs for both flow and level are output to a second file (DUPOUT). The simulated output hydrographs are given for those designated pipes i.e. pipes with a function flag number of 3 and most of the hydraulic structures such as storage tanks and CCOs, and also at the termination pipes. Samples of the two output data files are included in Appendices A(xiii) and A(xiv).

9.6 LIMITATIONS OF THE MODEL

The in-house model DUPPERS has been enhanced so that parallel pipe systems can be simulated. However, in common with many other computer simulation models, the enhanced DUPPERS model is based on

some assumptions and has its limitations. These are listed as follows :

(1) Imaginary manholes are set up at the upstream and downstream ends of the overflow pipe at cross-connections. This enables inflows and outflows to be stored for later hydrograph output before overflow rate is calculated. Any storage volume in these manholes cannot be accounted for.

(2) Bridging pipes are for flow transfer only and therefore no flow routing process is conducted in these pipes.

(3) Since there is no routing procedure in the bridging pipes, their length may not exceed the relatively small but arbitrary value of 10 times the pipe diameter.

(4) Head above a weir crest (Hav) is assumed to be constant along its length at any time interval.

(5) Direct inflows into the cross-connection manholes are not allowed, they have to be input to the pipe upstream.

(6) Two successive cross-connections have to be separated by at least one discrete pair of parallel pipes.

(7) Head losses at the cross-connection manholes are not taken into account unless flows in the incoming and outgoing pipes are pressurised.

(8) Currently only 10 cross-connections are permitted in the model.

The major enhancement to the DUPPERS model is the incorporation of the cross-connection component in the original model. Level computation, which is another major additional procedure, determines the flow regimes in the cross-connection and also identifies the mode of discharge in the connection i.e. free-surface, pressurised or transitional flows.

Despite the above limitations and assumptions, test systems were constructed to check the performance of the enhanced model components. Synthetic rainfalls were used to test the model performance under steady state condition. The parallel pipe model DUPPERS was then applied to the study system after it was found to be operating satisfactory. Details of the testing procedures and the corresponding output have been shown in Chapter 10.

SUBROUTINE	PURPOSE OF SUBROUTINE	ENHANCEMENT DETAILS
ENDORD	Reads in branch and pipe numbers and ancillary indices; computes and sorts out the end order number of each pipe	Modifies the program to recognise the twin pipe system and cross-connection, compute end order number for the twin pipes
DATCHK	Sorts and writes out the structure of the input system so that user can check if it is correct	Subroutine enhanced to check if parallel pipe and CCOs are input correctly and to send out error messages to output file
DATIN	Reads in the complete data for the full system and defaults global values to those not given in SSD file	Read in data for parallel pipes and CCOs, separates overflow and bridging pipes and relocates them accordingly, stores CCOS data for later use
CONST	Calculates & assigns constants to each pipe length, writes out table of derived constants to the check file	Calculate constants for parallel pipes and bridging pipes, and compute dry weather flows for twin pipes separately; major enhancement requires to incorporate level computation procedure
PROFIL	Calculates the 10 overland flow inlet hydrographs (3 for slope & 3 for PAPG and 1 for roof)	No enhancement required for this subroutine
IMPERV	Estimates distribution of % runoff for each of the 3 surface types and stores global % runoff	No enhancement required for this subroutine
FLOW	Main calculation routine to route flows through pipe network and output simulation results to output data file	Major enhancement performs here to route flows in the parallel pipes, also to identify the correct flow regimes in CCOs and to solve surcharged flow in parallel pipe system

TABLE 9.1 Descriptions Of The Major Subroutines In DUPPERS And The Details Of Enhancements

OVERFLOW TYPE	CONDITION STATEMENT [refers to Figure 9.7(a)]
(i)	YCO < STSL YCO > D3 YSW < CRSL
(ii)	YCO > STSL YCO > D3 YSW < CRSL
(iii)	YCO < STSL YCO > D3 YCO < CNSL YSW < CRSL BNSL ≤ YSW
(iv)	CNSL ≤ YCO YCO > D3 YSW < CRSL YSW < BNSL
(v)	CNSL ≤ YCO YCO > D3 YSW > BNSL either YSW < CRSL or CRSL < YSW < YCO

TABLE 9.2 Condition Statements For Free Surface And Pressurised Overflows

FLOW BEHAVIOUR	CONDITION STATEMENT [refers to Figure 9.7(b)]
Head Balance (vi)	YRLF = YRLS YCO > CNSL YSW > CRSL
Reverse Flow (vii)	YCO > CNSL YSW > CRSL YRLS > YRLF

TABLE 9.3 Condition Statements For Head Balance And Reverse Flow Conditions

BRANCH NUMBER	ANCILLIARY INDEX (NANI)	INDICATION
0 -- 499		Normal pipe (dendritic system)
500		Foul sewer (parallel pipe system)
600		Storm relief sewer (parallel pipe system)
700		Cross-connection overflow
800		Bridging pipe
	0 or blank	Normal pipe data
	2	Sewered sub-area
	3	Normal pipe data (with output hydrograph)
	4	Storm overflow
	5	Offline tank
	6	Online Tank

TABLE 9.4 Branch Number And Ancillary Index Values

BRANCH NUMBER	NATURE OF BRANCH	REQUIRED DATA IN SSD FILE
< 700	Normal pipe data	Sewer length, invert and ground levels, pipe diameter, contributing areas, % of paved & roofed areas, PAPG, SLOPE AND DWF
700	Cross-connection	Weir length, ground & weir crest levels, weir depth, weir type, user defined weir discharge coefficient (optional)
> 700	Bridging Pipe	Overflow branch and pipe numbers, pipe length, inlet & outlet invert levels, pipe diameter

TABLE 9.5 Details Of Required Data For Parallel Pipe System

NATURE	DESCRIPTION OF INPUT	COLUMN	FORMAT
PARALLEL PIPE (500, 600)	Branch label	1 - 3	integer
	Pipe label	5 - 7	integer
	Sewer ancillary index	8 - 9	integer
	Pipe length (m)	10 -14	integer
	Ground level at U/S manhole (m AD)	15 -21	decimal
	Upstream invert level (m AD)	22 -29	decimal
	Pipe diameter or top diameter for egg-shaped pipe (mm)	30 -34	integer
	Minor cross-section dimension (mm)	35 -40	integer
	Area contributing to pipe (ha)	41 -46	decimal
	Pipe shape index	47 -48	integer
	Number of extra manhole along sewer	49 -50	integer
	Index of manhole headloss	51 -52	integer
	Pipe roughness (mm)	53 -57	decimal
	Percentage of impermeable area	59 -60	integer
	Percentage of pitched roof area	62 -63	integer
	Percentage of flooded area	65 -66	integer
	Ground slope index	68	integer
	Paved area per gulley	69 -70	integer
	Dry weather flow to pipe (l/s)	74 -80	decimal
CROSS-CONNECTION OVERFLOW (700)	Overflow branch number	1 - 3	integer
	Overflow pipe number	5 - 7	integer
	Sewer ancillary index	8 - 9	integer
	Overflow pipe length (m)	10 -14	integer
	Ground level at overflow manhole	15 -21	decimal
	Upstream invert level (m AD)	22 -29	decimal
	Overflow weir depth (mm)	30 -34	integer
	Setting for overflow (l/s) (optional, do not enter if Cd used)	35 -40	integer
	Discharge coefficient for overflow weir (left blank for default value)	41 -46	decimal
	CCO type (0=type 1, 1=type 2)	47 -48	integer
BRIDGING PIPE (800)	Branch number	1 - 3	integer
	Branch number to where overflow goes	5 - 7	integer
	Pipe number to where overflow goes	8 - 9	integer
	Bridging pipe length (m)	10 -14	integer
	Upstream invert level (m AD)	15 -21	decimal
	Downstream invert level (m AD)	22 -29	decimal
	Pipe diameter (mm)	30 -34	integer

TABLE 9.6 Input Descriptions And Format For Parallel Pipes, CCOs And Bridging Pipes

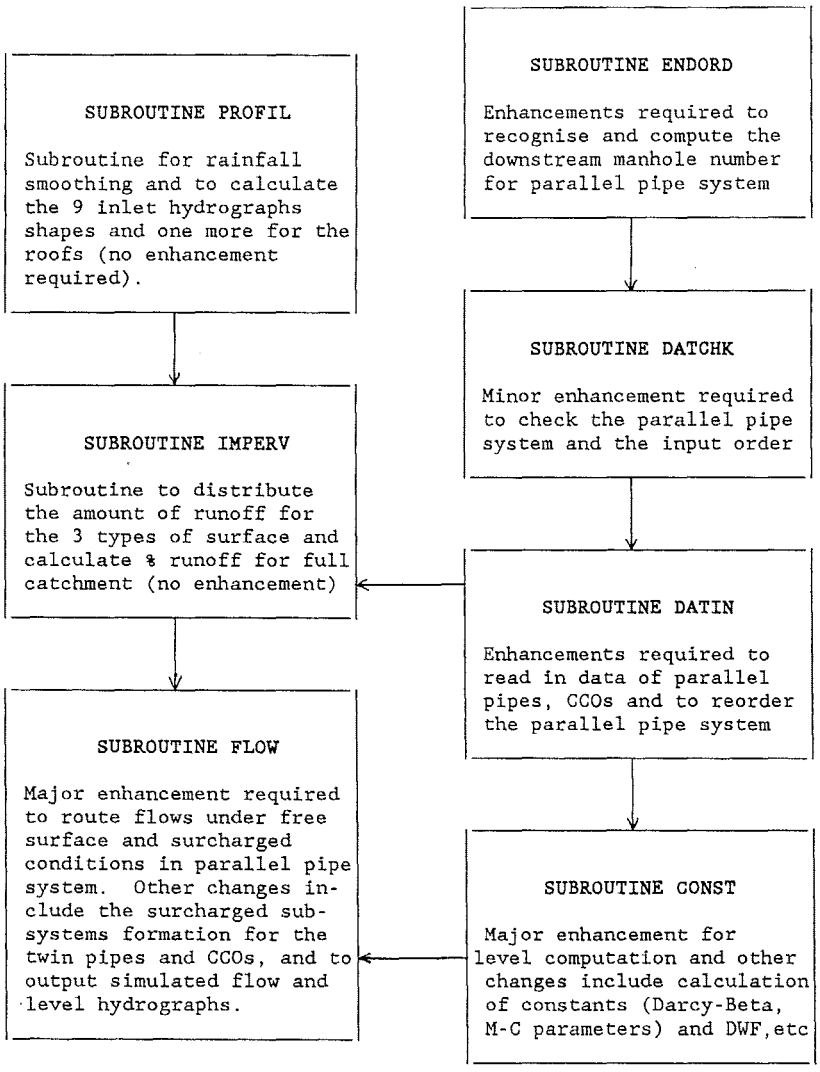
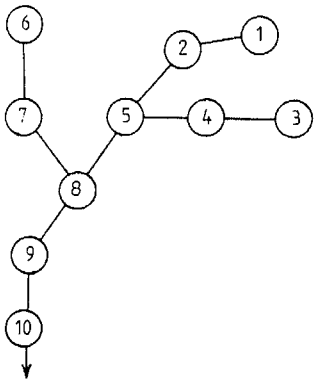
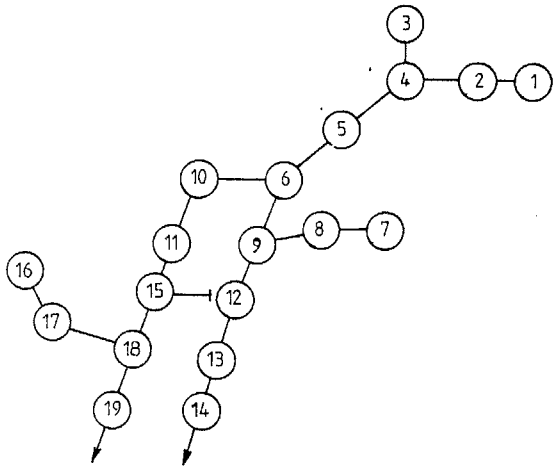


FIGURE 9.1 Flow Chart Showing the Enhancements Required for Each Major Subroutine in DUPPERS



(a) Downstream manhole numbers
for dentritic system (DUCTS)



(b) Downstream manhole numbers for parallel
pipe system (DUPPERS)

FIGURE 9.2 Downstream Manhole Numbers for Single and Parallel Pipe Systems

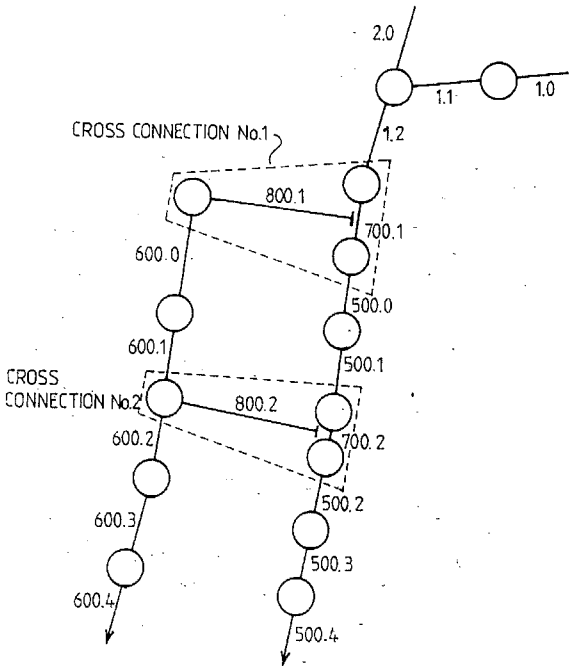


FIGURE 9.3 Branch and Pipe Numbering System for Parallel Pipes and Cross-Connections

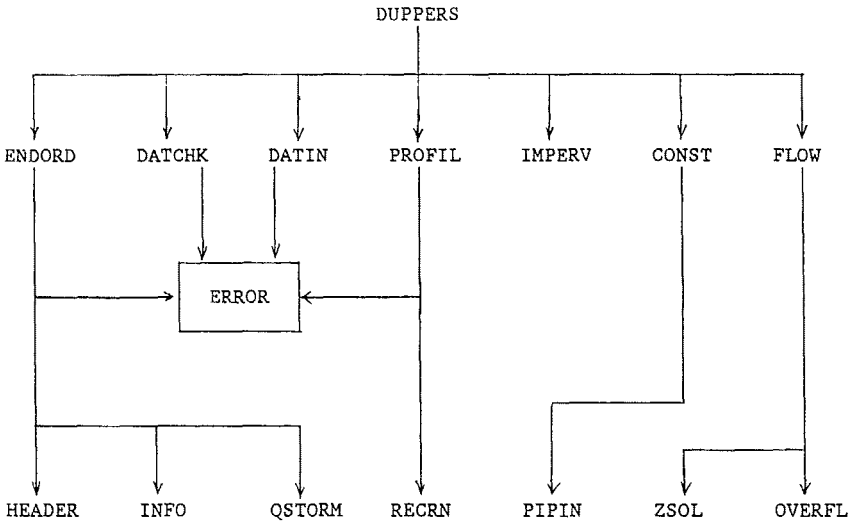
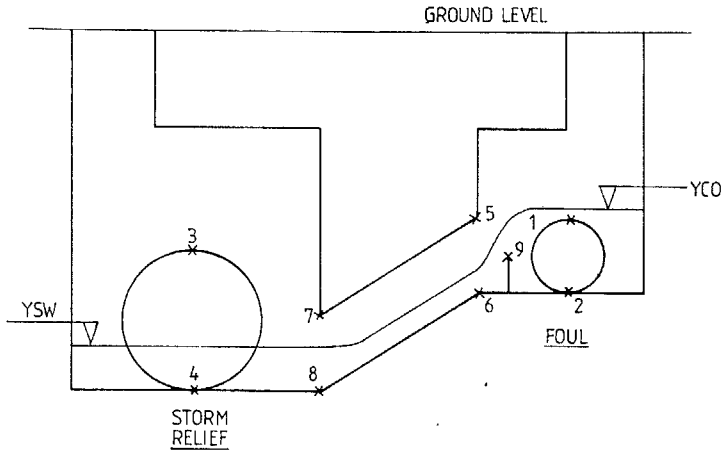


FIGURE 9.4 The Structure of Parallel Pipe Model DUPPERS

(7 major and 7 minor Computer Subroutines)



ARRAYS

1 STSL -- Foul outlet soffit level
2 STIL -- Foul outlet invert level
3 CRSL -- Storm relief outlet soffit level
4 CRIL -- Storm relief outlet invert level
5 CNSL -- Bridging pipe inlet soffit level
6 YCC -- Bridging pipe inlet invert level
7 BNSL -- Bridging pipe outlet soffit level
8 GCC -- Bridging pipe outlet invert level
9 D3 -- Weir crest level

FIGURE 9.5 Arrays Set-up for Storing Critical Locations in Cross-Connections

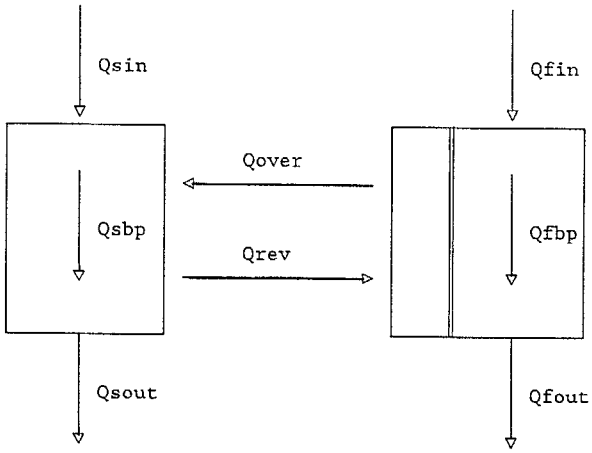
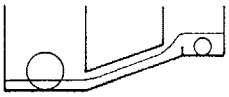
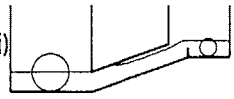
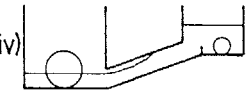
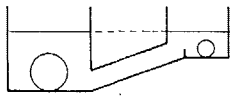
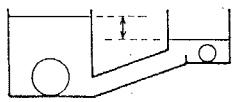


FIGURE 9.6 FLOW PATTERN FOR A TYPICAL CROSS-CONNECTION

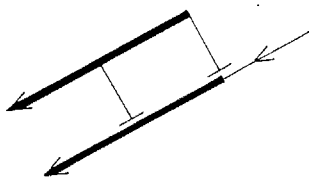
Flow patterns in cross connection	C.C. IN LYNEBURN SYSTEM				
	1	2	3	4	5
(i) 	YES	YES	YES	YES	YES
(ii) 	YES	YES	YES	YES	YES
(iii) 	YES	YES	YES	YES	YES
(iv) 	YES	YES	YES	YES	YES
(v) 	YES	YES	YES	YES	YES

(a) Overflow under free-surface and surcharged conditions

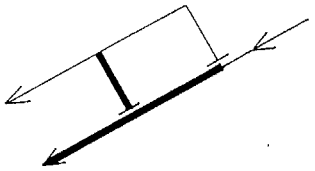
	NO	NO	NO	YES	YES
	NO	NO	NO	YES	YES

(b) Reverse and balance flow behaviours in cross-connection

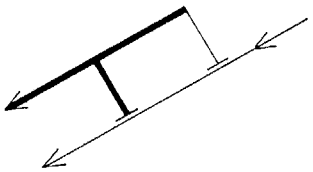
FIGURE 9.7 Possible Flow Behaviours in the Lyneburn Cross-Connection Overflows



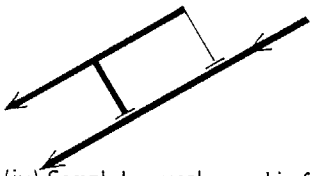
(i) Surcharged in twin pipes only



(ii) Foul and bridging pipes surcharged



(iii) Storm relief and bridging pipe surcharged



(iv) Complete surcharged in foul storm relief and bridging pipes

FIGURE 9.8 Possible Surcharged Sub-Systems along the Parallel Pipes and Cross-Connections

CHAPTER 10
SIMULATION MODEL ACCURACY ASSESSMENT

10.1 INTRODUCTION

To simulate a given sewerage system using standard models, accuracy is judged by the goodness of fit between the predicted results and observed data. Modifications to the constructed model are sometimes necessary in order to obtain a verified model without having to resort to force-fitting (Stickler 1986). Standard procedures on model testing and verification are widely available (WRC/WAA 1986, Eadon 1986, Price and Osborne 1986) but these are primarily to guide engineers on model building using standard packages. The process for model validation and verification on an urban sewer system has been discussed in the later section 10.4.

In order to demonstrate that the enhanced parallel pipe model was working satisfactorily, it was necessary to show that all sub-components adopted behaved correctly. It has been shown in Chapter 7 that the previously calibrated sub-models and the equivalent lumped pipe model suggest the overland and below-ground parameters have been identified and may readily be used as input data to the parallel pipe model. However, before the enhanced model could be applied to the Lyneburn system, the overflow procedures employed in the cross-connections and the flow routing in parallel pipes required to be tested and justified. The strategy used in parallel pipe model testing was firstly to check the assembled components and to show that the behaviour of these components was satisfactory when included in the Lyneburn parallel pipe model.

Validation and verification of the parallel pipe model were carried out using observed events. Graphical representation of the results are presented and compared with the observed data. Percentage differences when compared with observed values are presented.

10.2 INPUT DATA FOR TESTING

There are three fundamental hydraulic aspects of the new model which required to be tested :

(i) overflow computation,
(ii) flow quantities in the parallel pipes,
(iii) response of flows to different rainfall intensities,
(iv) level computation.

To meet all of the above, a testing strategy was set up utilising several known inflows into the parallel pipe system and determining corresponding amounts of overflow and outflow at cross-connections and outfalls respectively. Levels are also output together with the overflows and discharges in cross-connections in order to check the computing procedures.

10.2.1 TEST SYSTEMS

Two different types of 'idealised' parallel pipes systems were set up for the testing of components. These two systems are shown diagrammatically in Figure 10.1 and system details are given in Table 10.1. Both systems have the same contributing areas which are represented by three identical sewered sub-areas upstream of the parallel pipes. The first system (A) has one single cross-connection, whilst the second (B) has two cross-connections separated by three pairs of parallel pipes.

System A was assembled and used for refining algorithms during the model construction allowing remedies to be resolved more easily. It was then used to test the flow behaviour in the twin pipes and flow separation in the foul, storm relief and bridging pipes.

Apart from the testing of flow regimes in the two separate cross-connections, system B was used to identify the formation of surcharged sub-systems in the parallel pipes. Reduction of diameters of certain pipes in system B was made to allow the

treatment of the following to be examined :

(i) The behaviour of surcharge in any one of the two cross-connections or both,
(ii) Reverse flow and head balance at cross-connection,
(iii) Surcharged sub-systems consisting of either one or both cross-connections.

Table 10.2 gives details of the different pipe sizes in all test systems and the aims of each variation.

The structure used for testing overflow in the cross-connections was the type 1 single-sided weir detailed in Chapter 6. Figure 10.2 shows the configuration of such a connection used in both system A and B. Table 10.3 lists the diameters and hydraulic characteristics of the pipes, and the physical dimensions of the overflow weir used.

10.2.2 TEST INPUTS

The model used rainfall as test input. For testing of the hydraulic components, constant rainfall intensities were used with the idealised systems A and B. The rainfall intensities varied from 1.0 mm/h to 96.0 mm/h and each introduced a steady inflow to the test system through the sewered sub-areas. A total of eleven uniform rainfall intensities were chosen as listed in Table 10.4. The corresponding hand calculated and computer generated inflow values are also included in the table.

Small rainfall intensities of 3.0 to 15.0 mm/h were used to check the flows under free surface condition while the greater intensities from 36.0 mm/h were for testing pressurised flow in the twin pipes. Transitional flows were produced by those moderate rainfall intensities of 12.0, 15.0 and 18.0 mm/h.

Varying rainfall intensities were also used to test the idealised systems. The aims of using varying intensities was to check the stability of the model and the response of flows in the parallel pipes after a rapid increase or decrease of inflows. Figure 10.3

shows the rainfall hyetographs of the three varying intensities chosen. The first two types of varying rainfall started at small intensity with a peak of 36.0 mm/hr. The pattern was repeated three times with a total duration of 180 minutes. The third hydrograph was a single triangular type with zero rainfall for 30 minutes at the end used to check the flow behaviour in the parallel pipes after the peak with zero inflows.

10.3 TESTING COMPONENTS OF THE MODEL

The new components in the enhanced parallel pipe model require to be tested. The aim of testing is both to confirm the consistency of the flow behaviour in the cross-connections and to prove the robustness of the newly developed model. Contributing areas in the three sewered sub-areas and the overflow dimensions in systems A and B were kept constant and the pipe sizes were altered to create the flow regimes under both free surface and pressurised flows. Besides those already listed in Section 10.2.1, other testing aims were such as flow balance throughout the testing systems and prediction of head above weir crest.

Test system A with a single cross-connection was first used to check the free surface flows in the parallel pipes and flow continuity. System B with varying pipe diameters was then used to create the above flow regimes.

10.3.1 FREE SURFACE FLOWS

Flow modelling in the parallel pipe model consisted of two distinct parts. The first was to check the flow conditions for each pipe and later the surcharged pipes were identified, together with the extent of each surcharged sub-system. Flows in cross-connections were said to be free surface only if no flow paths were pressurised, including discharges in the foul and storm relief pipes, and across the overflow weir. Reverse flow under free

surface conditions, as demonstrated in Chapter 9, was not possible in all five cross-connections at one time and hence this flow phenomenon was not tested. Free surface flow tests were designed to check for :

(i) straight through flow only in the foul pipe without the occurrence of overflow,
(ii) overflow across the weir under free surface flow,
(iii) the difference between inflows and combined discharges at the outfalls, and
(iv) flow balance at cross-connections.

Six different rainfall intensities were used with system A and their outputs are summarised in Table 10.5 together with a comparison of the inflows and combined outflows. The first four tests showed that there was no flow in the storm relief pipe because the levels were lower than the weir crest and hence overflow did not occur. Overflow started once the inflow exceeded 187.88 l/s in the foul pipe and discharged into the storm relief pipe as shown. Discharge over the weir as shown in the overflow column in the table confirmed those in the storm relief, indicating that some discharge did end in the storm pipe. Two significant occurrences could be observed from these results. Any further increase of the inflow above 187.88 l/s resulted in the foul pipe becoming surcharged. Secondly, the foul pipe would become pressurised before the bridging pipe since the latter had a greater pipe-full capacity than the foul pipe.

After satisfactory responses were obtained from the output of system A, the System B was subjected to further investigation. The set-up of System B enabled the following to be examined :

(i) additional inflows from the upstream pipes to the second cross-connection,
(ii) direct inflow into the storm relief pipe before the overflow from the foul,
(iii) overflow occurrence at the second cross-connection.

The presence of the second cross-connection not only allowed the flow behaviour to be investigated, but also the computational

procedures and the employed hydraulic equations could be checked. Test procedures for system B were basically similar to those for system A. However, two slightly heavier rainfall intensities were used to cause overflow to occur in the second cross-connection. The test rainfall intensities varied from 1.0 mm/hr to 21.0 mm/hr as detailed in Table 10.4 earlier.

The eight steady state simulations covered three distinct overflow regimes in system B :

(i) no overflow in both cross connections,
(ii) overflow only in the upstream connection but not in the second,
(iii) overflow in both cross-connections.

All of the above took place with free surface flow only. Table 10.6 summarises the results for system B against the corresponding rainfall intensity. It can be seen from the results that the first six outputs were the same as for system A for the upper cross-connection. Both the test systems were hence said to be performing in a similar manner with the same inflows.

No overflow occurred for the inflows which were less than 131.45 l/s in the foul pipe in both cross-connections as seen in Table 10.6. Overflow in the first cross-connection started as inflow exceeded 187.88 l/s in the upper connection but not in the second, since levels at this weir were below the crest. Overflow then also took place in the lower cross-connection for inflow exceeding 200.65 l/s. All overflows caused by the test rainfall intensities occurred under free surface condition, i.e. overflows were less than the full-bore flow of the bridging pipes.

The above tests carried out for systems A and B enabled the checking of the flow in the parallel pipes and the flow computation in the cross-connections under free surface flow condition. The investigations showed that the following aspects of the model operated satisfactory :

(i) flow separation in the twin pipe system ;
(ii) continuity of flows at cross-connections and in the

parallel pipes ;

(iii) the flow determination procedures for cross-connections ;

(iv) there was a balance between the input discharge before the parallel pipe system and the final combined outflows.

10.3.2 PRESSURISED FLOW

For a parallel pipe system, the detection of surcharged mode and surcharged sub-systems in the twin pipes was essentially similar to that in a dendritic system. The checking procedures were carried out simultaneously for the entire parallel pipe system since there was a possibility of surcharging in one or more cross-connections. Once the pipe under consideration was identified as being surcharged, the checking procedures were extended to identify further pressurised pipes until a pipe downstream was determined to be flowing under free surface (Bettess & Price 1978). In this manner a sub-system was identified. Flows and appropriate heads were then determined for the entire sub-system.

Both partial and fully surcharged sub-systems which might form anywhere along the parallel pipe system had to be tested. Possible surcharging locations included :

(i) one or more foul pipes only,
(ii) one or more storm relief pipes only,
(iii) group of foul and bridging pipes,
(iv) group of foul, storm and bridging pipes.

The above cases were tested by alterations to pipe diameters. Table 10.2 gives details of the five test systems together with their objectives. System A was not used for this testing, because it had been included in the altered, new, double cross-connection test systems B1 to B5.

The technique for checking surcharged conditions was similar to the test procedures for the free surface flow component. Rainfalls of constant intensity were used with the idealised test system so that

surcharged sub-systems formed at the appropriate legs in the parallel pipes. The test intensities started at 36.0 mm/hr and increased to 96.0 mm/hr. The rainfalls used and their corresponding inflows are shown in Table 10.4.

Simulations have been carried out for all five variations to system B using the various rainfall intensities. Table 10.7 summarises the results together with the flowrates at the two cross-connections and the outfalls. The surcharged sub-systems formed in each test system are shown diagrammatically in Figure 10.4.

Test system B1 shows the foul pipe only surcharged. Surcharged flows firstly started in those foul pipes just upstream of the second cross connection as shown in (a) in Figure 10.4. With increased flow, the surcharged group extended to the entire length of foul pipes as in case (c). Testing systems B2 and B3, which have diameters reduced to 300mm and 225mm respectively, were surcharged in the lower leg of the foul pipe as shown in case (b). Increased inflow to system B3 also caused the second bridging pipe to be included in the surcharged group as in case (d).

In contrast to the above, test systems B4 and B5 had the storm relief pipe diameters reduced to 600mm and 450mm respectively. Results for system B4 indicated that it could be classified as being the same as B1. Surcharging occurred only in the foul pipe upstream from the second cross-connection. System B5, however, showed the entire parallel pipe system to be surcharged except for the lower leg of the foul pipes. A further flow phenomenon, head balance in both foul and storm relief manholes at the second cross-connection, was also demonstrated in system B5 at a high inflow. The problem associated with head balance is discussed in Section 10.3.6.

A comparison between the inflows and the combined outflows for the above test results were carried out and the results are shown in Table 10.8. The table shows that the outflows were slightly reduced when compared with the inflows. This was mainly due to the losses in the computational process (Ackers 1959) whilst the flow was

propogating downstream of a system (Ackers and Harrison 1964, Bettess and Price 1978). The difference between the inflows and outflows are less than 0.3% which is within the ±10% as stated by Ackers (1959) as seen in Table 10.8 and hence is considered to be very consistent.

From all the above results, it can be concluded that the pressurised flow component performed satisfactorily in steady state flows and predicted flows correctly in the parallel pipes. Furthermore, surcharged sub-systems were identified and resolved satisfactorily by the parallel pipe model. Continuity of flow was also preserved for the various types of flow regimes in the parallel pipe system.

10.3.3 IDENTIFICATION OF FREE SURFACE AND PRESSURISED FLOWS

Another flow condition in a sewer system is the transition between the part-full and surcharged flow. Initially at the start of an event the flow in all the pipes was assumed to be part-full with dry weather flow and the heads at all the manholes were zero. With varying inputs, discharges were calculated throughout the system at successive timesteps, each pipe being checked to determine whether the upstream discharge exceeded pipe-full discharge at each timestep. A group of surcharged pipes would then be identified and isolated to form a surcharged sub-system, the lower end point being determined when the pipe downstream from a surcharged pipe returned to the free surface flow condition.

The depth of flow under the free surface flow condition was determined according to the discharge in the pipe under consideration. As the flow depth increased to approximately 0.85 times the pipe diameter the transitional condition was approached. Previous research works suggested that flow in the transitional region is very unsteady (Barr 1980) and tends rapidly to become surcharged flow. It was hence important that the flow condition in cross-connections was determined because overflow was calculated based on the head above the weir. Transitional flow in the foul pipe in a cross-connection was classified as surcharged flow once the flow exceeded the downstream full-bore pipe.

Test results shown in both Tables 10.5 and 10.6 indicated that all flows in the cross-connections were under free surface, including flow over the weir, for inflows less than 233.47 l/s. Further increase in the rainfall intensities subsequently causing higher inflows to the system and flows in the foul pipe became surcharged as shown in Table 10.7. The enhanced parallel pipe model did identify the flow according to the discharge and level in the cross-connection. Procedures for determining the overflow were also correctly indentified even in the transitional region.

10.3.4 LEVEL PREDICTION

The level of flow was computed according to the discharge in a pipe at the manhole just upstream from that pipe and was output as level hydrographs in the results. In the part-full flow condition, the level was found using the relationships between the proportional flow and proportional depth in a pipe and assuming uniform flow. When a pipe surcharged, the depth in the upstream manhole might cause the next pipe upstream to surcharge.

The tests for the level predicting component were based on the tests for free surface and surcharged flows as shown in Tables 10.6 and 10.7. The predicted levels for the free surface flow using system B are shown in Table 10.9, whilst Table 10.10 gives the computed levels for the pressurised flow testing using the variously modified systems. Test system A was actually the same as the upper cross-connection in test system B and hence the predicted levels were the same as shown in Table 10.9.

Besides the computed figures, the flow depths found by hand calculation for both free surface and surcharge conditions have also been included in Tables 10.9 and 10.10 respectively. The free surface flow depth was found based on the discharges in parallel pipes together with the proportional depth and discharge relationship as shown in Chapter 8. For the surcharge flow, however, the depths in the manholes were calculated using the discharges in the manholes and the Darcy constants which were calculated for each pipe separately.

As seen from the tables, the percentage difference between the computed and hand calculated figures were less than ±10% (Williams 1985). The results indicate that levels were computed for the corresponding flow regimes in both the upper and lower cross-connections. Results from the system B5 with 54.0 mm/hr showed head balance occurred in both manholes. Further discussion of this particular flow behaviour is given in Section 10.3.6.

10.3.5 HEADS ABOVE WEIR

In a cross-connection, the foul through-flow and overflow at the weir have to be determined simultaneously. The computational procedures differ for determining the heads in the cross-connection for the part-full flow and pressurised flow conditions. In the free surface flow condition, the method of calculating the outgoing flow in foul was by using the flow-depth relationship (Ackers 1969, HRS 1983). Once the level was estimated from the proportional depth curve based on the incoming discharge, the level was also used to calculate the overflow across the weir, the computational details of which are discussed in the next section. The level estimated first was used as a start point for an iteration until the sum of the overflow and foul outgoing flow equalled the inlet flow upstream of the cross-connection. In the case of surcharge in the cross-connection manholes, the level was predicted simultaneously with any other inter-connected pipes in the surcharged flow sub-system. The computed levels in the surcharged cross-connections were again checked for consistency between the inflow and the combined outflows.

The three overflow types on the Lyneburn system and the corresponding hydraulic equations for computing the flow across their weirs have already been mentioned in Section 8.4.2. However, the test of the head above the weir crest, $H\omega$, was only carried out for the first overflow type shown in Figure 8.5, that is for free surface overflow. The other two were only taking place under surcharged flow condition and the heads used for computing the overflow were the total levels in the foul manhole.

The head over the weir crest was used in equations 8.17 to compute the amount of overflow across the weir. The same head over the weir was also used, according to the overflow type, to calculate the coefficient of discharge as in equations 8.22 and 8.23. The value of the discharge coefficient (Cd) was computed for each timestep, based on the new head above the weir which in turn was dependent on the incoming discharge.

The checking of the heads above the weir again made use of the previous test results, because the results for test system A had been covered in fact by those for system B and hence results were included in one table i.e. Table 10.11. In order to check that the model was predicting the correct overflow value under steady state conditions, overflows determined using hand calculation have also been included in the table and gave the comparison between the predicted and calculated values. It can seen from the table that the maximum difference for the two figures was only 1.2%. This result was sufficiently close and hence the overflow component was considered to be satisfactory.

The overflow across the weir was plotted in Figure 10.5 against the head above the weir for the results shown in Table 10.11. This graph shows that all overflows occurring in both cross-connections lie on a smooth curve against various heads above weir crest which in turn depended on the different rainfall intensities. The curve as shown on the graph was due to the head being raised by 1.5 times in the overflow equation shown also in Figure 10.5. This hence showed that the overflow was computed correctly using the employed overflow equation with the appropriate head.

10.3.6 OVERFLOW / REVERSE FLOW AND HEAD BALANCE

Under normal circumstances there was no overflow from the foul to the storm relief pipes at the beginning of a simulation. As the storm flow increased, overflow commenced once the level in the foul manhole was higher than the crest of the weir. Three different overflow modes were possible as shown in Figure 8.5. Condition A represented overflow occurring mostly in the beginning or the end

of a storm. Condition B occurred when the foul manhole was surcharged and was the most common type during moderate events. The last condition C only occurred when both pipes were surcharged and the head in the foul pipe was higher than in the storm relief manhole.

All the above overflow conditions have been incorporated in the model. The test for the first overflow mode has been discussed in Section 10.3.5 and the results are grouped in Table 10.6. Overflow under the pressurised conditions B and C, however, can be seen in Table 10.7. The computed overflow rates for the different modes have been demonstrated to be working satisfactorily.

Close investigation into the study catchment (see Chapter 9) suggested that the most downstream cross-connection in the parallel pipe system was frequently subjected to a balance of heads and reverse flow conditions during heavy events, demonstrating the need for a head balance check in the test schedule. A balance of heads, as defined in Chapter 9, in the two manholes of this particular cross-connection may occur after a rapid increase of inflow in the storm relief pipe during extreme events. The flow in the storm relief pipe is spilled from the four upstream cross-connections. The velocity of flow reduces as the flow reaches the mild slope at the lowest cross-connection and the level quickly increases. Further increase of flow in the storm relief manhole after the head balance results in having the flow travelling in a reverse direction into the foul manhole.

The head balance test has been covered in the test of the pressurised flow component. The occurrence of head balance can be seen in Table 10.7 and Table 10.10 shows the two levels in both the foul and storm manholes respectively in the last test. Figure 10.6 shows the head balance phenomenon diagrammatically.

Reverse flow did not normally occur in the study system unless flow in both the parallel pipes had become pressurised and a further increase in level in a storm relief manhole occurred after the

heads had become equal. The occurrence of reverse flow was checked for by the model whenever the head in the storm relief manhole was greater than in the foul. The computation procedures for reverse flow have been reviewed in Chapter 9.

To determine the model performance under reverse flow conditions, an additional test system B6 had to be set up by further alterations to System B. Details of the alterations required are shown in Table 10.12. A simulation was carried out for only the heaviest test rainfall intensity 96.0 mm/hr. The corresponding simulation outputs are also summarised in Table 10.12 and the results demonstrated that flow could return to the foul from the storm relief pipe in the lower cross-connection. The hand-calculation checks on the computed reverse flow value are also shown in Table 10.12.

10.3.7 CONTINUITY OF DISCHARGE

Continuity of discharges must be maintained for all the above flow regimes. The testing of the continuity of discharge required an investigation of the balance of flows in the entire parallel pipe system. Checks were also required to quantify the amount of flow reduction after overflow from the foul pipe, and the summation of discharge in storm relief pipe, and exceptionally vice versa for the case of reverse flow in the cross-connections.

The test results in Tables 10.5, 10.6 and 10.7 give the discharges not only in the parallel pipe system, but also the inflows just before the start of the twin pipes. The outputs show that the combined outflows through the final outfalls were close to the inflows before the parallel pipes. The test results for all system B are summarised in Table 10.13 to show the discharges within the systems and to observe the continuity of discharge for all test systems under various rainfall intensities. The results demonstrate that continuity of flow was preserved throughout.

10.3.8 VARYING INTENSITY RAINFALLS

The stability and response of the enhanced parallel pipe model were tested using the varying rainfall intensities shown in Figure 10.3. Details of the three different types of rainfalls and their aims can be summarised as :

(i) 3.0 mm/hr for 30 minutes and sudden increase to 36.0 mm/hr for a further 30 minutes, repeated 3 times. Used to observe the flow response of the model for a sudden increase and decrease of the input discharges in the parallel pipe system ;

(ii) Triangular rainfall profile starting with 3.0 mm/hr and increasing to 36.0 mm/hr linearly in 30 minutes, followed by a linear reduction to 3.0 mm/hr over another 30 minutes, repeated 3 times. For testing the flow behaviour in parallel pipes as the inflow rises and falls gradually ;

(iii) A single triangular profile with rainfall starting from nothing and rising to 36.0 mm/hr linearly, and falling back to 0 mm/hr over 70 minutes. To check whether the flow returns to the base flow condition in the twin pipes after the peak of a storm, by providing a long duration of zero input discharge after a gradual recession of input.

Simulations were carried out for the above rainfall profiles with three test systems chosen from Table 10.2. These systems were B1, B3 and B5. The output hydrographs obtained for the foul and storm relief outfalls are plotted together in figures 10.7, 10.8 and 10.9.

Similar responses were obtained for both systems B1 and B5 for all the three varying rainfalls. It can be seen from Figure 10.7 that the response of models to the rapid inflow to the parallel pipe system was in the same manner as the rainfall profile. The output also indicated that the flow in the storm relief pipe started rapidly as overflow commmenced and returned to no flow as overflow ceased. System B3, however, had much higher flow in the storm

relief than in foul due to the small outfall diameter, i.e. 225mm. The small diameter pipe acted as a throttle and was pressurised during most of the simulation and hence most of the flow passed over to the storm relief pipe.

Flow responses with all test systems for the second varying rainfall were similar. It can be noticed from Figure 10.8 that the storm outfall hydrographs were triangular in shape as the rainfall profile. Again the hydrograph for the foul pipe of system B3 was similar to that in Figure 10.7 and also subjected to surcharged flow during simulation. Figure 10.9 shows flow hydrographs obtained using the last varying rainfall. Results are similar to those in Figure 10.8. Figures 10.7 to 10.9 also show that all the hydrographs return to the base flow for the foul pipes and to zero flow in the storm relief pipe.

The results obtained using the varying rainfall intensities demonstrated that the aims of testing as set out in the beginning of this section had been met.

10.3.9 CONCLUSIONS FROM MODEL COMPONENTS TESTING

Using the idealised test systems together with the artifical uniform rainfall intensities, flow behaviour in cross-connections and parallel pipes could easily be observed. The performance of the components in the enhanced model was also being investigated under steady state conditions and the following was concluded from the tests above :

(i) Flow regimes were correctly identified in the parallel pipes and cross-connections ;
(ii) Flow conditions were clearly distinguished between free surface and pressurised flows ;
(iii) Levels were found to be correctly computed for both free surface and surcharged flows in the new model ;
(iv) Flows were conserved throughout the test systems as shown in the flow continuity section ;
(v) Flows over the weir were computed correctly using the overflow equation and the computed heads above the weir ;

(vi) All possible flow behaviour could be identified in a cross-connection including overflow, reverse flow and head balance ;

(vii) Various surcharged sub-systems along the parallel pipe system could be formed by the enhanced model and this included the presence at cross-connections.

Apart from all the above, the tests using varying rainfall profiles showed that the model responded correctly to the various inflows into the parallel pipe systems. With different configurations of the test systems, various amounts of discharge in the twin pipes were computed and these have been shown in Figures 10.7, 10.8 and 10.9.

After the components in the enhanced parallel pipe model had been tested for their accuracy and performance, the model was then used on the global Lyneburn parallel pipe model.

10.4 PERFORMANCE OF THE LYNEBURN SIMULATION MODEL

The purpose of the sewer simulation modelling was to define the state of service and limitations of the performance of a sewer system. Safety margins for example, expressed in terms of return periods for surface flooding, can be ascertained by model testing using statistically predicted rainfall patterns. Remedial measures and enhancement to safety margins, can be implemented optimally by testing a model with various operational or constructional changes until the most cost effective method is found.

The normal process of sewer simulation modelling is shown diagrammatically in Figure 10.10. In addition to the normal simulation model setting up procedures, the process for the DUPPERS model construction is also outlined in this figure and has been detailed in Section 10.4.2. Essentially this is based on the setting up of a numerical (computer) representation of the details of a sewer system, including sewer sizes, lengths and contributing drainage areas. This model is then fed with rainfall information

of specified intensity and duration, and the details of the resultant hydraulic effects on the system in the form of rates of flow and water levels are computed at sequential time intervals by the computer software.

As with any modelling process, the sewer flow simulation model has to undergo a 'proving' process. This was two stage and entailed :

(i) VALIDATION ---- the adjustment required to ensure that the computer model estimated accurately the flow behaviour at cross-connections, and

(ii) VERIFICATION -- by comparision of modelled hydraulic behaviour with that observed from flow monitors installed at strategic points in the sewer system.

Urban sewer flow modelling requires a considerable investment in surveys both to define the system details and also to monitor the flows and levels in the system in response to rainfall. System details and the overland characteristics have also been described fully in Chapters 5 and 6. Contributing drainage areas to the Lyneburn system have been verified by the lumped pipe model using the standard commercial software WASSP (Chapter 7). These verified system data for both overland and below ground were used on the validation and verification of the enhanced software DUPPERS.

The model of the full Lyneburn parallel pipe system was constructed with its lowest point being the Bothwell Street overflow chamber. Prior to model simulation using real events, its performance under steady state condition was determined. The tests performed with uniform rainfall intensities enabled the following to be identified :

(i) Overall performance of the assembled parallel pipe model ;

(ii) Sensitivity of flows and overflow at cross-connections ;

(iii) Flow behaviour in the twin pipes and cross-connections under steady state conditions.

None of the above would be easily observed using real storms. The constant rainfalls used for the Lyneburn model were similar to those for testing the constructed components.

Three models, each with an outfall progressively further downstream, were built for the Lyneburn parallel pipe system :

(i) Rex Park,
(ii) Mill Road, and
(iii) Bothwell Street.

These three locations correspond to locations where flow data had been monitored. To calibrate the above models, two events each were used for the Rex Park and Mill Road sub-catchments and six for the Bothwell Street catchment. The performance of the models was then verified by using the same number but separate events. Details of the events used for calibration and verification are shown in section 10.4.2.

The simulation outputs for all events used are summarised in tables and also in hydrograph format for easy comparison.

10.4.1 MODEL PERFORMANCE UNDER STEADY STATE CONDITIONS

Uniform rainfall intensities are an effective means of testing model components as shown in Section 10.3. To test the performance of a complete model under steady state conditions, constant rainfalls have also been used. Unlike those used for the component testing, the constant rainfalls chosen for the steady state checks have intensities of 1, 5, 10, 15, 20, 25 and 30 mm/hr. Each was used for simulation with the Bothwell Street parallel pipe model.

Simulation outputs for all rainfall intensities are plotted as hydrographs in Figures 10.11(a) and 10.11(b) for foul and storm relief pipe respectively and the predicted peak discharges in the twin pipes for each simulation are summarised in Table 10.14. The values of the peak flowrates for the corresponding intensities are plotted in Figures 10.12(a) and 10.12(b).

Figures 10.11(a) and 10.11(b) indicate that both the flows in the parallel pipes were directly proportional to the rainfall intensities, as expected. However, the rate of increase of flow in the foul pipe reduced as the rainfall intensity approached 30.0 mm/hr. In contrast, flows in storm relief pipe increased rapidly, indicating that large amounts of flow passed over the weirs at the five cross-connections. It can also be seen that a progressive gradual increase in flows and this could be due to the hidden contributing areas (see Chapter 7).

Figures 10.12(a) and 10.12(b) give a representation of flow behaviour in the parallel pipe model. The peak discharges in the foul pipe (Figure 10.12(a)) rose almost linearly until the rainfall reached approximatly 20.0 mm/hr and then increased at a reduced rate. Similar behaviour can be seen in Figure 10.12(b) as peaks in storm relief pipe increased rapidly but slowed down as the intensity approaches 20.0 mm/hr. This linear rise in the graphs showed that overflows were occurring in direct proportion to the rainfall and hence the inflows to the parallel pipe system. The rates of increase of the peaks reduced in both graphs, mainly caused by surface flooding at various locations along the system.

The peak discharges in both the foul and storm relief pipes for all the uniform rainfall intensities are given in Table 10.14. The combined peak discharges for all the storms are also included, together with the percentages of peaks to the combined peak for the twin pipes. A graph showing the percentage of foul and storm peaks to the combined against inflows is plotted in Figure 10.13. It can be seen that the ratio of peaks to the combined peaks for the foul pipe reduced as the inflows into the system increased. Conversely however, the ratio of peaks in the storm pipe increased steadily as the inflows increased. It was also interesting to note that 34% of the combined peak flow was already present in the storm relief sewers for the smallest rainfall intensity of just 1.0 mm/h, and up to 83% for the heavy rainfall of 30.0 mm/h. This observation in the peak discharges not only suggested that overflows were taking place in the cross-connections, but also indicated that the storm relief sewers were taking most of the storm flow during events. Furthermore, the rapid decrease in the peak discharge and early

overflow between 1.0 and 5.0 mm/h rainfalls in the foul pipe suggested that the 'first foul flush' could be passed over to the storm relief and cause the pollution in the receiving watercourse even for small events.

The simulation outputs and the analysis of the results showed that the model was performing satisfactorily under steady state conditions. By inputing various rainfall intensities, the parallel pipe model predicted the amount of discharge separately for the twin pipes proportional to the inflows. The steady state hydrographs showed also the flows would rise steadily in the twin pipes and recess after the storms. Overflows were also found to steadily increase as shown by the peak discharges in the storm relief pipes in Figure 12(b). The parallel pipe model constructed was considered by these results to be verified and performing satisfactorily. They showed that it could be used for model calibration and verification with real events.

10.4.2 DUPPERS MODEL CALIBRATION AND VERIFICATION

To construct a theoretical model for an urban drainage system, it is normally required to be calibrated by identifying the contributing areas, overland characteristics and sewer diameters etc and then further to be verified. In constructing the DUPPERS model, however, the calibration and verification procedures were somewhat different to the normal stages, as shown in Figure 10.10. The Lyneburn system was firstly calibrated and verified by building a lumped pipe model (Chapter 7) in order to define the catchment area boundaries and identify the contributing areas. These verified system data were then used on the new model. For the model DUPPERS, calibration was the stage to identify the program errors and to amend the computer codings (Figure 10.10). The check on the parallel pipes and cross-connection input data in the SSD file was also in the calibration stage. This calibrated model was then further verified in order to justify its working standard.

With the availability of three separate flow monitoring locations along the Lyneburn parallel pipe system, three computer models could hence be constructed. The flow monitoring locations and

their catchment boundaries have already been shown in Figure 5.8 in Chapter 5. The smallest model for the Rex Park catchment was firstly calibrated and each further model became a development of the smaller one. Checking and calibration was completed on the most upstream catchment before progressing downstream. This procedure thus enabled discrepancies to be resolved most efficiently.

Model calibration and verification are the necessary processes to demonstrate the performance of the constructed model (Clifforde & Green 1984, Green & Drinkwater 1985, Ashley 1986). It is hence particularly important to note that two separate batches of observed events were prepared, one for model calibration and the other for the model verification (Williams 1984). The observed events were separated into two categories according to their intensities and rainfall duration :

(i) high intensity with short duration, and
(ii) low intensity with longer duration.

The events chosen for calibration and verification are listed in Table 10.15 in chronological order. The table gives the details of the events monitored together with their duration, rainfall depth and the average intensities. The peak rainfall intensities for a particular event are also given as these should correspond with the peak runoff in the simulation output of the model. Some of these events were used in two models since flow data was available at more than one location. Furthermore, the simulation output could be compared at the two locations for the same event.

Table 10.16 lists all the storms against the three locations and their modelling purposes. Two events each were considered to be sufficient for the calibration of the small and medium sub-catchments, Rex Park and Mill Road. However, six had to be taken for the more complex global Bothwell Street catchment. The events for calibration and verification for the three models were mixed in nature, with both high and low intensities with different duration as shown in the categories above.

One of the options in using simulation packages is either an user input value for the standard percentage runoff (SPR) or the computer defaulted figure using standard equation (Jefferies and Ashley 1986, HRS 1985). As already discussed in Chapter 6, percentage runoff was one of the important parameters indicating the amount of runoff in proportion to the rainfall. For the purpose of calibration and comparison, SPR values as defined in Chapter 6 for the three locations were used for half of the ten simulations. For the other five calibration events, the computer default figures were used. Table 10.16 indicates the SPR values for the three models and those events simulated with the SPR figures.

Simulations were performed for all ten calibration events with the three models. Flow hydrographs from three of these are shown together with the observed data in Figures 10.14, 10.15 and 10.16. The full set of ten hydrograph outputs are listed and included in Appendix C.

Figures 10.14 and 10.16 are the simulated outputs from Rex Park and Bothwell Street respectively with the use of SPR values. Figure 10.15 is for Mill Road with the default SPR value of 45%. The events used with the Rex Park and Mill Road models were multi-peaked rainfalls while for Bothwell Street was a single peak storm. All three outputs showed that close fits were obtained for both predicted and observed flows. Fuller investigations and analysis have been carried out and are reported as in the following sections.

As the models had been calibrated, they were then subjected to the verification process. All simulations were performed with the user input SPR values in the verification process as those ones used in model calibration. The flow hydrographs were plotted for the three models. Again only one each for the Rex Park and Mill Road catchments as shown in Figures 10.17, 10.18, but two for Bothwell Street in Figures 10.19 and 10.20. The ten full sets of hydrographs are again shown in Appendix C.

The two events used for simulations are shown in Figures 10.19 and 10.20 for the Bothwell Street catchment and are the same as used for Rex Park and Mill Road in Figures 10.17 and 10.18 respectively. Both storms were multi-peaked, having heavy intensities. The predicted flow hydrographs shown in the figures were a close match to the observed data and the predicted peak discharges were also close to the observed values. Comparisons of the peak flowrates, runoff volumes and percentage runoff values are included in later sections.

The observed flow data for the event of 17th June 1986 (Figure 10.19) were only available for part of the whole event in the foul pipe at Rex Park due to a logger malfunction. However, a close match has been achieved for the first 300 minutes and the last peak discharge was predicted by the model. Close fit was also obtained between computed and observed hydrographs for the storm relief pipe throughout the complete event where data was available and the model was considered to have been verified in spite of the missing data. The percentage difference between the observed and predicted values of peak discharge and runoff volume is presented in the next section.

10.5 ANALYSIS OF MODEL PERFORMANCE

The accuracy of a simulation model depends both on the input data and the constructed model. For the parallel pipe model, the amount of discharge through the final outfalls depended on the overflow split at the cross-connections. It can be seen from the hydrographs that a good match was achieved between the observed data and the modelled results. However, some of the storms were over-predicted especially for the Rex Park catchment as seen in Appendix B. The discrepancies are due to :

(i) errors in the observed data,

(ii) observed peak discharges missing, due to data logging at 30 minutes intervals,

(iii) limitations and approximations inherent in the enhanced model.

To investigate the general performance of the model and to analyse the computed results, all simulated results including those shown in Appendix B were used. WRc (Williams 1985) recommends that verification can be considered to be achieved when :

(i) timings agree to ±5 minutes

(ii) volumes are within ±20%, and

(iii) peak flows agree to ±20%.

The hydrographs show that the predicted peak flows agreed with the observed peaks and the time was within ±5 minutes for most of the simulations except those when the peaks were logged at 30 minutes time intervals. As seen from the plotted hydrographs, the predicted and observed hydrographs are very close to each other in both the peak discharges and the runoff volumes. The computed runoff volume, peak discharge and level required to be further analysed and compared with the observed values in order to achieve the model verification and the details are enclosed in the following sections.

10.5.1 PEAK DISCHARGE

The predicted peak discharges are tabulated together with the observed values in Tables 10.17 and 10.18. Table 10.17 shows results for both the Rex Park and Mill Road whilst 10.18 contains those for the Bothwell Street only. Percentage difference between the observed and modelled peaks are also included in the tables.

The results show that the percentage difference for the computed and observed peaks were all within the ±20% with a range from -16.1% to +4.7%. The differences also indicated that most of the peak flowrates were over-simulated, but only by a small amount.

The peak flows in the tables are used to plot as graphical forms and shown in Figures 10.21, 10.22 and 10.23 for Rex Park, Mill Road and Bothwell Street respectively. Each figure shows the foul and storm relief flows separately.

All points in the graphs are close to the 45^0 line and this is particularly the case for the storm relief flows. The Bothwell Street catchment (Figure 10.23) shows a better representation of the peaks since more events are available for simulation.

The computed peak flows were also plotted against the total rainfall depth as shown in Figures 10.24, 10.25 and 10.26 for the three catchments. The points are scattered more or less randomly because more than one peak is present for most storms. However, a significant observation could be made from these graphs that the cluster of points for the foul occurred at much lower peak discharges than in the storm relief pipe. This indicated that much higher peak flows occurred in the storm relief than in the foul pipe as rainfall increased.

Due to the fact that a peak flow was caused by a corresponding peak rainfall intensity in a storm, a further set of graphs was plotted for the three catchments. The peak rainfall intensities for storms are listed in Table 10.15 corresponding to the peak flows in Tables 10.17 and 10.18. The resulting graphs are included as Figures 10.27, 10.28 and 10.29.

Figure 10.27 shows that the points are scattered for both the twin pipes. Those for the Mill Road in Figure 10.28, however, showed that the one peak intensity could produce a range of peak discharges. Flows at these two locations were clearly affected by the rainfall types and duration. The peak flows at Bothwell Street parallel pipes, however, had a better correlation with the peak intensities as shown in Figure 10.29. This graph also showed that peak flows in the storm relief pipe increased directly to the rainfall intensities.

It can be seen from the above comparison that both the observed and computed peaks were within ±20% range (Tables 10.17 and 10.18) and

hence the peak flows computed by the model were acceptable. Furthermore, the graphical comparisons showed good correlations between peak flows in the parallel pipes and rainfalls at different locations and therefore the peak discharges predicted by the model were satisfactory.

10.5.2 RUNOFF VOLUME IN PARALLEL PIPES

Runoff volumes were determined from the hydrographs for all simulations. The observed and predicted runoff volumes for the parallel pipes were tabulated in Tables 10.19 and 10.20 for Rex Park, Mill Road and Bothwell Street respectively. The percentage differences between observed and predicted were all within ±20% for the runoff volumes at Bothwell Street, but one each for Rex Park and Mill Road exceeded the -20%. This was due to the data being monitored in 30 minutes intervals and hence peak flows probably being missed out.

The data are plotted in Figures 10.30, 10.31 and 10.32 for the three catchments. It can be seen that most points are close to the 45^0 line except some in the foul pipes, the discrepancies being due mainly to the logging time difference mentioned above. Correlation between the computed and observed was better for the runoff volumes in the storm relief pipe. More events were available for Bothwell Street and therefore correlation was apparently better than for the other two catchments.

Dry weather flows were included in the computed runoff volumes for the three models in Tables 10.19 and 10.20. To investigate the relationships between the runoff volume and the rainfall characteristics, the nett runoff volume was required. The dry weather flows in the parallel pipes and the sizes of the three catchments were shown in Table 10.21. Nett runoff volumes were then calculated for the three catchments and tabulated in Tables 10.22 and 10.23.

Separate graphs were then plotted using the calculated nett runoff

volumes and these are :

(i) Nett runoff volumes in foul and storm relief against rainfall depth,

(iii) Combined nett runoff in twin pipes against rainfall depth.

The above graphs were only plotted for the Bothwell Street catchment because only four points were available for Rex Park and Mill Road catchments. The three graphs were included in Figure 10.33. As seen in graph (a), a higher runoff volume with lower rainfall depth was possible in the foul pipe. This was probably the result of low rainfall intensity with long duration events. However, most of the other points in the graph were close to the 'best straight' line. Correlation was again better in the storm relief pipe indicating increased runoff volume with rainfall depth. In the combined runoff volume (graph (c) in Figure 10.33), the cluster of points locate at the similar positions for the storm relief pipe as in graph (b). This was due to the fact that flow in the storm relief was much higher than in the foul, dominating the position of the points.

Analysis of the runoff volumes in the parallel pipes indicated that the amount of runoff in the foul depended on the rainfall type and duration. Flows in the storm relief pipe, however, suggested that the amount of overflow was directly proportional to the rainfall depth and not governed by the rainfall type for the range of events tested. The close percentage difference, except for a few events in the foul pipe, between the predicted and observed runoff volumes, indicated that the model performed satisfactory.

10.5.3 RUNOFF VOLUME BETWEEN VARIOUS LOCATIONS

The objectives for checking the runoff volumes at different locations was not only to determine the ratio of flow in foul and storm relief pipes but also to establish the amount of flows between two locations along the parallel pipe system for the three computer models.

The ratio of volume between the foul and storm relief flows was only determined for the Bothwell Street location because storms were insufficient for Rex Park and Mill Road sub-catchments. The net flow volumes are summarised in Table 10.24, event by event. The minimum was found to be 0.24 with the maximum being 1.27. The ratio was irregular being due to the foul flow not being proportional to the rainfall intensity. However, an average of 67.5% was found for the volume of flow in foul to the storm relief flow.

The flow difference between two locations was also determined. The amount of flow volumes at the three locations should increase downstream in the parallel pipes during a storm since each catchment includes the last one upstream. To investigate the flow difference at these locations, the flow volumes in the twin pipes were combined. The combined runoff volumes for the three locations were tabulated event by event in Table 10.25. The percentage values shown in the table are the difference of runoff volumes for that location compared with the Bothwell Street catchment total runoff. The average percentage difference in runoff between Mill Road and Bothwell Street is 60.5%. The difference between Rex Park and Bothwell Street is found to be 72.6%. This higher percentage difference could possibly be due only having two available events for Rex Park.

The derived percentage difference in runoff volumes between different locations was compared with the percentage difference in catchment size between the Rex Park, Mill Road to Bothwell Street. The catchment size difference between Rex Park and Bothwell Street is 46.8% whilst the runoff volume is 72.6%. Similarly, the catchment different between Mill Road and Bothwell Street is 59.9% and 60.5% for the difference of combined runoff between these two locations. The figures indicated that the difference in flow volume between Mill Road and Bothwell Street was very small and it was concluded that catchment size and the runoff volumes in the twin pipes were proportional to each other.

10.5.4 PERCENTAGE RUNOFF

Percentage runoff indicates the amount of outflow in relation to the rainfall depth for a catchment. Percentage runoff values have been established using the observed data as discussed in Chapter 6. With the flows now predicted by the enhanced parallel pipe model, the percentage runoff values were determined using the simulated results. The percentage runoff values derived in this method for the three locations were then compared with those in Chapter 6.

The net combined runoff volumes for Rex Park, Mill Road and Bothwell Street are tabulated in Table 10.26 together with the values of RUNVOL. Graphs were then plotted with the RUNVOL against the rainfall depth (P) for the three locations and these are included as Figures 10.34, 10.35 and 10.36. Best straight line using linear regression method was then drawn for each graph in order to establish an equation in the form of :

$$RUNVOL = RC \cdot (P - DEPSTOG)$$

The runoff coefficient (RC) is the slope of the line and depression storage (DEPSTOG) is the intercept point on the rainfall axis. The appropriate constants are shown on the figures. These values are summarised in Table 10.27 and compared with those determined with observed data in Chapter 6. Error in the percentage runoff is slightly higher for Rex Park but closer for the other two catchments. Depression storage showed good correlation for all three locations.

The computed RUNVOL values for the three locations are plotted on the same graph as shown in Figure 10.37 in order to derive an average routing coefficient and depression storage constants for the global Lyneburn catchment. The average RC and DEPSTOG were found to be 0.45 and 3.028mm respectively from the graph, indicating that the enhanced model gave the average percentage runoff of 45% for the Lyneburn catchment with 3.03mm depression storage. As a comparison with the observed values, the predicted 45% for RUNVOL was found to be close to the Mill Road location (40% shown in Table 10.27) but slightly low for Rex Park and Bothwell

Street. However, the average DEPSTOG 3.03 agreed with all three observed figures as shown in Table 10.27. From the comparison as shown, the enhanced model was considered to be predicting close percentage runoff and depression storage constants for the catchment.

10.5.5 LEVEL OF RELIABILITY

The computed levels given in the simulation outputs required to be checked against observed data. To determine the accuracy of the predicted levels, three level hydrographs were plotted, one each for the three locations.

Computed data for three separate events are included in Figures 10.38, 10.39 and 10.40 together with the observed values. A good match was obtained between the predicted and observed levels for all catchments. The computed peak levels are also close to the observed data.

The computed and observed peak levels from these events are summarised in Table 10.28. The percentage difference between the observed and predicted values ranges from -4.9% for the Rex Park catchment to +2.2% for Bothwell Street and well within the ±20% band as suggested by WRc. The predicted and observed levels given in Table 10.28 are plotted separately for foul and storm relief and shown in Figure 10.41. All the plotted levels on the graph (a) in Figure 10.41 lie close to the 45^0 lines except one for Rex Park location. Similarly, the levels for the storm as shown in graph (b) also has a close correlation between the observed and predicted values.

As shown from the above comparisons, the levels computed by the enhanced model were close to the recorded values and hence the computational procedures for predicting level was performing satisfactorily. Furthermore, the close match between the observed and computed level hydrographs validated the newly developed depth component in the enhanced DUPPERS computer model, which had not existed in its predecessor DUCTS model.

10.6 SUMMARY

The enhanced parallel pipe DUPPERS model has been constructed. The degree of robustness of the model had been identified by testing individual components using artificial rainfalls. The overall performance of the model was then verified using real events and compared with the observed data. The performance of each component was checked, mainly by comparing the computed values with the hand calculated figures, and the percentage of difference was then found for these two. To determine the response of flows under steady state condition in parallel pipes, the enhanced model was further tested using uniform rainfalls before being verified using real storms.

After the components had been tested using various types of artificial events, the aims of testing as listed in the beginning of this chapter was fulfilled. The following give the achievement from the process of component testing :

(i) Employed hydraulic equations as shown in Chapter 8 are found to be correctly adopted in the enhanced model and hence predict accurate overflows, through-flows and reverse flows ;

(ii) The flow paths in cross-connections have been correctly identified by the new model, which in turn will determine the directions of overflow and reverse flow ;

(iii) DUPPERS distinguishes between free surface and surcharged flows in parallel pipes and hence determines the appropriate overflow or reverse flow equations ;

(iv) Surcharge sub-systems are correctly formed in the parallel pipe system with or without the presence of cross-connections ;

(v) Levels are correctly computed for both free surface and pressurised conditions for which different computational procedures will be used.

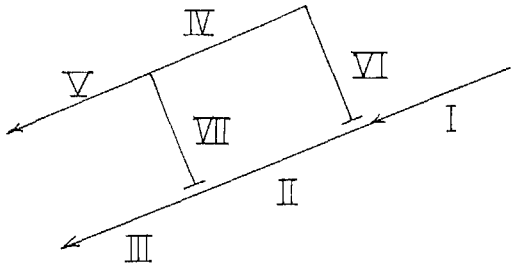
The model predicts flows and levels in the parallel pipes under steady state conditions. Enhanced model also shows overflows

discharge into storm relief sewers for rainfalls as small as 1.0 mm/h. This early overflow behaviour, as indicated by the model (Table 10.14), causes a rapid increase of flow in the storm relief pipe even for small and moderate events. This premature high flow in the beginning of an event enables the first foul flush pollutants to be discharged from the foul to the storm relief. The model then proceeds to identify the behaviour of head balance and reverse flow conditions and to give good predictions for both.

In simulating the study catchment with real events, the model predicted flows at the outfalls for the parallel pipes having a very close match with the observed data. The model also predicted the missing flows for the foul pipe and confirmed the flows in storm relief for the 17th June event at Rex Park. Further investigation and analysis of the computed results also confirms the satisfactory performance of the model. Predicted catchment constants such as PR, DEPSTOG and RC all agreed with those derived from the observed data. Level, as computed by the new component in the model, is also found to agree with the recorded data and to have a good fit with the recorded level hydrographs.

DETAILS OF TEST SYSTEM	TESTING PIPE SYSTEM	
	SYSTEM A	SYSTEM B
Number of sub-catchments (SSAs)	3	3
Number of flow-carrying pipes before parallel pipe system	2	2
Number of parallel pipes (including bridging pipe)	9	14
Number of cross-connections	1	2
Number of outfalls	2	2

TABLE 10.1 Details of the Two Testing Parallel Pipe Systems



Parallel Pipe Test System B
(Note : II, III, IV and V represent the full leg of pipes)

PIPE SYSTEM	PARALLEL PIPES				PURPOSES OF TESTING
	FOUL		STORM		
	II	III	IV	V	
B1	450	450	800	800	Overflow in VI and VII under free-surface and surcharged flow conditions in foul pipes ;
B2	600	300	800	800	
B3	600	225	800	800	Continuity of flow at cross-connections ;
B4	450	450	600	600	Effects of surcharging in pipes IV, VI and VII ;
B5	450	450	450	450	Heads balance and reverse flow at lower cross-connection.

TABLE 10.2 Pipe Diameters and Corresponding Test Objections

PHYSICAL & HYDRAULIC CHARACTERISTICS COMPONENT		DETAILS OF THE PARALLEL PIPES AND CROSS-CONNECTION								
		FOUL				STORM RELIEF			BRIDGING PIPE	OVERFLOW WEIR
Diameter	(mm)	225	300	450	600	450	600	800	450	/
Length	(m)	100	100	100	100	100	100	100	3.0	/
Gradient	(%)	1.0	1.0	1.0	1.0	1.0	1.0	1.0	10.0	/
Full Bore Capacity	(l/s)	36	78	230	497	230	497	1071	730	/
Roughness	(mm)	0.3	0.3	0.3	0.3	6.0	6.0	6.0	0.3	/
Weir Length	(m)	/	/	/	/	/	/	/	/	1.0
Weir Height	(m)	/	/	/	/	/	/	/	/	300

TABLE 10.3 Dimensions and Hydraulic Details of Parallel Pipes and Cross-Connections in Test Systems A and B

UNIFORM RAINFALL INTENSITY (mm/h)	CORRESPONDING INFLOWS FROM THREE SSAs TO THE PARALLEL PIPE SYSTEM		PERCENTAGE DIFFERENCE (%)
	HAND CALCULATED (l/s)	COMPUTED (l/s)	
1.0	15.5	16.1	+3.7
3.0	46.5	46.6	+0.2
6.0	93.0	89.1	-4.4
9.0	139.5	131.5	-6.1
12.0	186.0	187.9	+1.0
15.0	232.5	233.5	+0.4
18.0	279.0	279.3	+0.1
21.0	325.5	326.4	+0.3
36.0	558.0	559.2	+0.2
54.0	837.0	835.8	-0.2
96.0	1488.0	1446.7	-2.9

TABLE 10.4 Uniform Rainfall Intensities and Corresponding Inflows to the Parallel Pipe Systems

RAINFALL INTENSITY (mm/h)	INFLOW BEFORE PARALLEL PIPES (l/s)	FLOWRATES @ CROSS-CONNECTION			DISCHARGES AT OUTFALLS			PERCENTAGE DIFFERENCE BETWEEN INFLOW AND OUTFLOW (%)
		FOUL (l/s)	STORM RELIEF (l/s)	OVERFLOW (l/s)	FOUL (l/s)	STORM RELIEF (l/s)	COMBINED (l/s)	
1.0	16.11	16.11	0.0	0.0	16.02	0.0	16.02	0.6
3.0	46.61	46.61	0.0	0.0	46.34	0.0	46.34	0.6
6.0	89.07	89.07	0.0	0.0	88.92	0.0	88.92	0.2
9.0	131.45	131.45	0.0	0.0	131.28	0.0	131.28	0.1
12.0	187.88	183.56	4.32	4.32	183.31	4.28	187.59	0.2
15.0	233.47	188.62	44.85	44.85	188.57	44.77	233.34	0.1

TABLE 10.5 Simulated Outputs for Test System A under Free-Surface Flow Conditions

RAINFALL INTENSITY (mm/h)	INFLOW BEFORE PARALLEL PIPE SYSTEM (l/s)	DISCHARGES AT CROSS-CONNECTIONS						FINAL OUTFALLS		
		UPPER CROSS-CONNECTION			LOWER CROSS-CONNECTION			FOUL (l/s)	STORM RELIEF (l/s)	COMBINED DISCHARGE (l/s)
		FOUL (l/s)	STORM RELIEF (l/s)	OVERFLOW (l/s)	FOUL (l/s)	STORM RELIEF (l/s)	OVERFLOW (l/s)			
1.0	16.11	16.11	0.00	0.00	16.05	0.00	0.00	15.86	0.00	15.86
3.0	46.61	46.61	0.00	0.00	46.47	0.00	0.00	46.31	0.00	46.31
6.0	89.07	89.07	0.00	0.00	88.92	0.00	0.00	88.71	0.00	88.71
9.0	131.45	131.45	0.00	0.00	131.29	0.00	0.00	131.16	0.00	131.16
12.0	187.88	183.56	4.31	4.31	183.44	4.25	0.00	183.29	4.17	187.46
15.0	233.47	188.62	44.85	44.85	188.56	44.68	0.00	188.43	44.52	232.95
18.0	279.32	213.74	65.58	65.58	200.65	78.43	13.01	200.46	78.32	278.78
21.0	326.37	237.16	89.21	89.21	208.04	118.13	28.97	207.85	118.02	325.87

TABLE 10.6 Simulation Outputs for Test System B under Free-Surface Flow Condition

TEST SYSTEM REFER. NUMBER	RAINFALL INTENSITY (mm/h)	INFLOW BEFORE PARALLEL PIPE SYSTEM (l/s)	DISCHARGES AT CROSS-CONNECTIONS						FINAL OUTFALLS		
			UPPER CROSS-CONNECTION			LOWER CROSS-CONNECTION			FOUL (l/s)	STORM RELIEF (l/s)	COMBINED DISCHARGE (l/s)
			FOUL (l/s)	STORM RELIEF (l/s)	OVERFLOW (l/s)	FOUL (l/s)	STORM RELIEF (l/s)	OVERFLOW (l/s)			
B1	36.0	513.79	253.62*	260.17	260.17	203.95	309.55	49.53	203.56	309.27	512.83
B1	54.0	663.40	288.36*	375.04	375.04	225.87	437.03	62.09	225.33	436.86	662.19
B1	96.0	728.67	326.25*	402.42	402.42	242.52*	485.72	83.57	241.98	485.36	727.34
B2	36.0	513.79	386.38	127.41	127.41	91.89*	421.06	293.77	91.42*	420.83	512.25
B2	54.0	663.40	462.86	200.54	200.54	111.32*	551.58	351.14	111.06*	551.37	662.43
B3	18.0	279.32	279.32	0.00	0.00	38.98*	240.08	240.11	38.33*	240.02	278.35
B3	96.0	728.67	517.13*	211.54	211.54	78.98*	649.21	437.84	78.35*	649.03	727.38
B4	54.0	663.40	288.36*	375.04	375.04	225.87	437.04	62.11	225.35	436.86	662.21
B5	54.0	663.40	299.21*	364.19*	364.19*	228.75	434.98*	69.83*	228.45	433.73*	662.18

* denotes Surcharged Flow

TABLE 10.7 Simulated Outputs for the Five Dual Overflow Test Systems with Various Rainfall Intensities

TEST SYSTEM REFERENCE NUMBER	RAINFALL INTENSITY (mm/h)	INFLOWS TO THE PARALLEL PIPE SYSTEM (l/s)	COMBINED DISCHARGES AT OUTFALLS (l/s)	PERCENTAGE DIFFERENCE (%)
B1	36.0	513.79	512.83	0.2
B1	54.0	663.40	662.19	0.2
B1	96.0	728.67	727.34	0.2
B2	36.0	513.79	512.25	0.3
B2	54.0	663.40	662.43	0.1
B3	18.0	279.32	278.35	0.3
B3	96.0	728.67	727.38	0.2
B4	54.0	663.40	662.21	0.2
B5	54.0	663.40	661.18	0.3

TABLE 10.8 Percentage Difference between Inflows and Combined Outflows for Surcharge Component Test

RAINFALL INTENSITY (mm/h)	INFLOW BEFORE PARALLEL PIPE SYSTEM (l/s)	COMPUTED LEVEL FOR PARALLEL PIPE TEST SYSTEM B							
		UPPER CROSS-CONNECTION				LOWER CROSS-CONNECTION			
		Foul		Storm Relief		Foul		Storm Relief	
		Computed (m)	Hand Cal (m)	Computed (m)	Hand Cal (m)	Computed (m)	Hand Cal (m)	Computed (m)	Hand Cal (m)
1.0	16.11	0.081	0.0804	0.0	0.0	0.079	0.0784	0.0	0.0
3.0	46.61	0.132	0.1314	0.0	0.0	0.128	0.1273	0.0	0.0
6.0	89.07	0.193	0.1894	0.0	0.0	0.189	0.1863	0.0	0.0
9.0	131.45	0.245	0.2418	0.0	0.0	0.241	0.2389	0.0	0.0
12.0	187.88	0.318	0.3164	0.048	0.0461	0.311	0.3088	0.045	0.0436
15.0	233.47	0.381	0.3794	0.112	0.1116	0.372	0.3711	0.108	0.1068
18.0	279.32	0.405	0.4043	0.141	0.1402	0.336	0.3356	0.149	0.1479
21.0	326.37	0.426	0.4254	0.168	0.1674	0.364	0.3633	0.188	0.1868

TABLE 10.9 Predicted Levels for Test System B under Free-Surface Flow Conditions

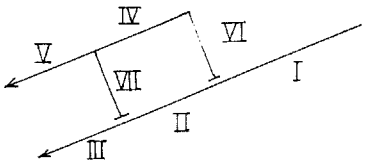
TEST SYSTEM REFERENCE NUMBER	RAINFALL INTENSITY (mm/h)	INFLOW BEFORE PARALLEL PIPE SYSTEM (l/s)	COMPUTED LEVEL FOR PARALLEL PIPE TEST SYSTEM B							
			UPPER CROSS-CONNECTION				LOWER CROSS-CONNECTION			
			Foul		Storm Relief		Foul		Storm Relief	
			Computed (m)	Hand Cal (m)	Computed (m)	Hand Cal (m)	Computed (m)	Hand Cal (m)	Computed (m)	Hand Cal (m)
B1	36.0	513.79	0.5477	0.5463	0.2721	0.2716	0.3968	0.3954	0.3011	0.3003
B1	54.0	663.40	0.6121	0.6116	0.3203	0.3194	0.4381	0.4374	0.3528	0.3517
B1	96.0	728.67	0.6270	0.6264	0.3384	0.3378	0.4862	0.4857	0.3762	0.3754
B2	36.0	513.79	0.4609	0.4589	0.1842	0.1833	0.5684	0.5676	0.3480	0.3475
B2	54.0	663.40	0.5108	0.5093	0.2325	0.2318	0.5926	0.5918	0.4008	0.4000
B3	18.0	279.32	0.3183	0.3177	0.0000	0.0000	0.5358	0.5350	0.2522	0.2517
B3	96.0	728.67	0.6012	0.6008	0.2392	0.2387	0.6408	0.6399	0.4417	0.4411
B4	54.0	663.40	0.6121	0.6116	0.3903	0.3897	0.4381	0.4374	0.4321	0.4315
B5	54.0	663.40	0.9872*	0.9861	1.7872*	1.7860	0.5982	0.5975	1.8685	1.8677

* Head balance phemonemon (see Figure 10.6)

TABLE 10.10 Predicted Levels for Various Altered Test Systems under Surcharged Flow Conditions

RAINFALL INTENSITY (mm/h)	INFLOW BEFORE PARALLEL PIPE SYSTEM (l/s)	CROSS-CONNECTIONS IN TEST SYSTEM B							
		UPPER CROSS-CONNECTION				LOWER CROSS-CONNECTION			
		Head Above Weir Crest (m)	OVERFLOW Computed (l/s)	OVERFLOW Hand Cal (l/s)	% Diff (%)	Head Above Weir Crest (m)	OVERFLOW Computed (l/s)	OVERFLOW Hand Cal (l/s)	% Diff (%)
1.0	16.11	0.000	0.00	0.00	0.0	0.000	0.00	0.00	0.0
3.0	46.61	0.000	0.00	0.00	0.0	0.000	0.00	0.00	0.0
6.0	89.07	0.000	0.00	0.00	0.0	0.000	0.00	0.00	0.0
9.0	131.45	0.000	0.00	0.00	0.0	0.000	0.00	0.00	0.0
12.0	187.88	0.018	4.32	4.37	-1.2	0.000	0.00	0.00	0.0
15.0	233.47	0.081	44.85	44.92	-0.2	0.000	0.00	0.00	0.0
18.0	279.32	0.105	65.58	65.61	-0.1	0.036	13.09	13.14	-0.4
21.0	326.37	0.126	89.21	89.25	-0.1	0.064	28.97	29.01	-0.2

TABLE 10.11 Heads above Weir Crest with Corresponding Overflows at the Two Cross-Connections for Test System B



Numbering for Altered Test System

LOCATION ALONG SYSTEM	PIPE SIZE AND HYDRAULIC DETAILS	COMPUTED FLOWRATES (l/s)
I	Inflows from three sewered sub-catchments	728.67
II	Upper leg of foul pipe diameter = 225 mm	101.21
III	Lower leg of foul pipe diameter = 800 mm	466.82
IV	Upper leg of storm relief diameter=450 mm	626.33
V	Lower leg of storm relief diameter=450 mm	261.85
VI	Bridging pipe diameter = 450 mm ; Darcy Beta constant (β) = 19.55	627.46 (overflow)
VII		365.61 (reverse)

CHECK FOR COMPUTED REVERSE FLOW AT LOWER CROSS-CONNECTION :

$\because$ Hfoul = 1.1289m and Hstorm = 3.7421m

Hence Hdiff = 3.7421 − 1.1289

= 2.6132m

$\because$ β for bridging pipe = 19.55

$\therefore$

$$Qrev = \sqrt{\left(\frac{2.6132}{19.55}\right)}$$

= 0.3656 m^3/s

or Qrev = 365.6 l/s (c.f. 365.61 l/s as VII above)

TABLE 10.12 Details of Altered Parallel Pipe System and Computed Reverse Flow together with Hand-Calculation Check

TEST SYSTEM REFER. NUMBER	RAINFALL INTENSITY (mm/h)	DISCHARGES @ VARIOUS LOCATIONS ALONG PARALLEL PIPES * I (l/s)	II (l/s)	III (l/s)	IV (l/s)	V (l/s)	VI (l/s)	VII (l/s)
B	1.0	16.11	16.05	15.86	0.00	0.00	0.00	0.00
	3.0	46.61	46.47	46.31	0.00	0.00	0.00	0.00
	6.0	89.07	88.92	88.71	0.00	0.00	0.00	0.00
	9.0	131.45	131.29	131.16	0.00	0.00	0.00	0.00
	12.0	187.88	183.56	183.29	4.31	4.25	4.31	0.00
	15.0	233.47	188.62	188.43	44.85	44.52	44.85	0.00
	18.0	279.32	213.74	200.46	65.58	78.32	65.58	13.01
	21.0	326.37	237.16	207.85	89.21	118.02	89.21	28.97
B1	36.0	513.79	253.62	203.56	260.17	309.27	260.17	49.53
	54.0	663.40	288.36	225.33	375.04	436.86	375.04	62.09
	96.0	728.67	326.25	241.98	402.42	485.36	402.42	83.57
B2	36.0	513.79	386.38	91.42	127.41	420.83	127.41	293.77
	54.0	663.40	462.86	111.06	200.54	551.37	200.54	351.14
B3	18.0	279.32	279.29	38.33	0.00	240.02	0.00	240.11
	96.0	728.67	517.13	78.35	211.54	649.03	211.54	437.84
B4	54.0	663.40	288.36	225.35	375.04	436.86	375.04	62.11
B5	54.0	663.40	299.21	228.45	364.19	433.73	364.19	69.83

* Refer to Table 10.2 for the locations in parallel pipe system

TABLE 10.13 Continuity of Flow along the Parallel Pipes for the Double Cross-Connections Test Systems

UNIFORM RAINFALL INTENSITY (mm/h)	PEAK DISCHARGES IN PARALLEL PIPES			RATIO OF FLOW TO THE COMBINED	
	FOUL (l/s)	STORM RELIEF (l/s)	COMBINED (l/s)	FOUL (%)	STORM RELIEF (%)
1.0	237.60	124.63	362.23	65.6	34.4
5.0	419.63	831.13	1250.76	33.6	66.4
10.0	580.66	1563.65	2144.31	27.1	72.9
15.0	701.16	2862.24	3563.40	19.7	80.3
20.0	796.54	3604.71	4401.25	18.1	81.9
25.0	887.73	4272.94	5160.67	17.2	82.8
30.0	918.82	4581.36	5500.18	16.7	83.3

TABLE 10.14 Peak Discharges and Ratio of Flows in Parallel Pipes under Steady State Condition

EVENT DATE	DURATION (min)	RAINFALL DEPTH (mm)	MEAN RAINFALL INTENSITY (mm/h)	PEAK RAINFALL INTENSITY (mm/h)
860130	776	8.2	0.634	3.0
860510	312	14.4	2.769	7.5 12.0
860512	291	5.7	1.175	3.0
860517	404	11.2	1.663	6.0
860610	608	16.9	1.668	6.0
860612	612	6.2	0.608	6.0 3.0
860617	700	32.6	2.794	30.0 27.0 9.0
860806	532	9.1	1.026	4.5
860813	248	5.9	1.427	7.5
860815	760	14.2	1.121	6.0 27.0
860816	300	11.3	2.260	9.0 7.5
860902	556	18.1	1.953	4.5 7.5 7.5
861019	228	8.1	2.132	10.5 6.0
861019(B)	176	5.0	1.705	6.0 7.5

TABLE 10.15 Details of Events for Simulation

Note : Two or more numbers in final column for each event indicate multi-peaked events.

EVENT DATE	PARALLEL PIPE CATCHMENT		
	REX PARK	MILL ROAD	BOTHWELL STREET
860130	/	/	CALIBRATION (SPR)
860510	CALIBRATION (SPR)	/	CALIBRATION (DEF)
860512	CALIBRATION (DEF)	/	/
860517	VERIFICATION	/	/
860610	/	/	CALIBRATION (SPR)
860612	/	/	VERIFICATION
860617	VERIFICATION	/	VERIFICATION
860806	/	VERIFICATION	VERIFICATION
860813	/	CALIBRATION (SPR)	CALIBRATION (DEF)
860815	/	/	VERIFICATION
860816	/	CALIBRATION (DEF)	CALIBRATION (SPR)
860902	/	VERIFICATION	VERIFICATION
861019	/	/	VERIFICATION
861019(B)	/	/	CALIBRATION (DEF)

NOTE : SPR -- User Input SPR Value
DEF -- Computer default SPR value

TABLE 10.16 Events and the Purpose of Modelling for the Three Parallel Pipe Catchments

CATCHMENT	EVENT DATE	FOUL			STORM RELIEF		
		OBSERVED PEAK (l/s)	COMPUTED PEAK (l/s)	PERCENTAGE DIFFERENCE (%)	OBSERVED PEAK (l/s)	COMPUTED PEAK (l/s)	PERCENTAGE DIFFERENCE (%)
REX	860510	193.30	193.54	-0.1	991.50	1009.89	-1.9
		220.90	225.67	-2.2	1372.50	1392.55	-1.5
	860512	156.80	164.93	-5.2	86.80	98.79	-13.8
	860517	191.10	206.88	-8.3	428.80	467.74	-9.1
PARK	860617	180.03	186.26	-3.5	1913.60	1922.89	-0.5
		214.27	213.58	+0.3	1321.60	1337.89	-1.2
		239.67	250.82	-4.7	892.38	905.57	-1.5
			Average = -3.4			Average = -6.1	
MILL	860806	109.82	120.66	-9.9	466.81	485.12	-3.9
	860813	120.52	132.88	-10.3	564.70	581.96	-3.1
	860816	129.04	149.79	-16.1	1150.60	1170.68	-1.7
		115.33	118.32	-2.6	920.71	939.31	-2.0
ROAD	860902	99.12	110.83	-11.8	408.35	410.54	-0.5
		131.25	143.09	-9.0	950.92	992.52	-4.4
		117.99	124.36	-5.4	743.11	825.70	-11.1
			Average = -9.3			Average = -3.8	

TABLE 10.17 Observed and Computed Peak Discharges in Parallel Pipes for Rex Park and Mill Road Catchments

EVENT DATE	FOUL			STORM RELIEF		
	OBSERVED PEAK (l/s)	COMPUTED PEAK (l/s)	PERCENT DIFF. (%)	OBSERVED PEAK (l/s)	COMPUTED PEAK (l/s)	PERCENT DIFF. (%)
860130	300.79	327.31	-8.8	549.87	581.81	-5.8
860510	480.52	495.81	-3.2	970.51	924.87	+4.7
	680.63	701.11	-3.0	1866.10	1836.35	+1.6
860610	311.46	327.32	-5.1	875.43	871.24	+0.5
860612	199.54	205.87	-3.2	490.26	503.82	-2.8
	136.09	147.06	-8.1	111.79	125.53	-12.3
860617	536.80	561.85	-4.7	4048.90	4365.27	-7.8
	705.80	742.02	-5.1	3325.10	3568.92	-7.3
	379.71	415.87	-9.5	2012.00	2275.24	-13.1
860806	295.82	313.87	-6.1	574.52	602.13	-4.8
860813	329.03	349.84	-6.3	694.44	703.10	-1.2
860815	374.67	383.52	-2.4	694.35	726.25	-4.6
	386.62	403.12	-4.3	834.29	835.45	-0.1
860816	396.93	425.86	-7.3	1622.10	1639.54	-1.1
	264.52	277.85	-5.0	1189.60	1220.87	-2.6
860902	218.99	242.15	-10.6	483.07	523.88	-8.4
	254.64	281.29	-10.5	1176.00	1225.33	-4.2
	229.24	243.74	-6.3	969.82	982.41	-1.3
861019	399.88	439.30	-9.9	801.72	834.24	-4.1
	315.67	313.62	+0.6	731.55	749.78	-2.5
861019 (B)	284.80	293.79	-3.2	591.07	614.67	-4.0
	276.50	279.59	-1.1	467.74	492.87	-5.4
		Average=	-5.6		Average=	-3.9

TABLE 10.18 Observed and Computed Peak Discharges in Parallel Pipes for Bothwell Street Catchment

CATCHMENT	EVENT DATE	FOUL			STORM RELIEF		
		OBSERVED RUNOFF (m^3)	COMPUTED RUNOFF (m^3)	PERCENTAGE DIFFERENCE (%)	OBSERVED RUNOFF (m^3)	COMPUTED RUNOFF (m^3)	PERCENTAGE DIFFERENCE (%)
REX	860510	5439.26	4524.52	16.8	13937.02	11697.27	16.1
	860512	2732.60	2609.14	4.5	684.35	969.26	-41.6
	860517	4025.82	3672.41	8.8	3724.74	3948.02	-6.0
PARK	860617	6809.55	6745.64	0.9	26771.49	23599.31	11.8
MILL	860806	2369.40	2145.07	9.5	3120.64	2875.70	7.8
	860813	1651.12	1609.85	2.5	2649.48	3439.88	-29.8
	860816	2628.75	2129.39	19.0	8718.42	7933.94	9.0
ROAD	860902	3062.24	2808.76	8.3	9467.58	8088.72	14.6

TABLE 10.19 Observed and Predicted Total Runoff Volumes in Parallel Pipes for Rex Park and Mill Road Catchments

EVENT DATE	FOUL			STORM RELIEF		
	OBSERVED RUNOFF (m^3)	COMPUTED RUNOFF (m^3)	PERCENT DIFF. (%)	OBSERVED RUNOFF (m^3)	COMPUTED RUNOFF (m^3)	PERCENT DIFF. (%)
860130	14684.97	12413.29	15.5	9096.83	8000.68	12.0
860510	9757.18	9005.05	7.7	11194.26	9757.05	12.8
860610	7908.35	7711.45	2.5	14114.11	13476.67	4.5
860612	4497.20	4086.20	9.1	3588.78	3380.25	5.8
860617	13566.18	13354.62	1.6	51338.29	47065.47	8.3
860806	4542.94	4257.99	6.3	4560.13	4205.02	7.8
860813	3443.72	3275.82	4.9	3462.99	3727.01	-7.6
860815	7686.70	7225.25	6.0	7754.36	7592.36	2.1
860816	6549.89	6123.00	6.5	11237.05	10563.77	6.0
860902	5529.02	5946.60	-7.6	14733.62	14711.52	0.2
861019	6187.40	5741.73	7.2	4496.56	4670.39	-3.9
861019 (B)	3810.86	3769.39	1.1	3765.11	3837.11	-1.9

TABLE 10.20 Observed and Predicted Total Runoff Volumes in Parallel Pipes for Bothwell Street Catchment

CATCHMENT	SIZE OF CATCHMENT (ha)	DRY WEATHER FLOW	
		FOUL (l/s)	STORM RELIEF (l/s)
REX PARK	175.095	35.852	0.00
MILL ROAD	224.078	44.156	0.00
BOTHWELL STREET	374.040	50.721	20.33

TABLE 10.21 Catchment Sizes and Dry Weather Flows for Three Parallel Pipe Catchments

CATCHMENT	EVENT	COMPUTED RUNOFF VOLUME		NETT RUNOFF VOLUME*	
		FOUL (m^3)	STORM (m^3)	FOUL (m^3)	STORM (m^3)
REX	860510	4524.52	11697.27	3042.40	11697.27
	860512	2609.14	969.26	1595.96	969.26
	860517	3672.41	3948.02	2648.48	3948.02
PARK	860617	6745.64	23599.31	4388.01	23599.31
MILL	860806	2145.07	2875.70	682.62	2875.70
	860813	1609.85	3439.88	637.53	3439.88
	860816	2129.39	7933.94	592.76	7933.94
ROAD	860902	2808.76	8088.72	1288.03	8088.72

* Nett Runoff Volume = Computed Runoff Volume - DWF

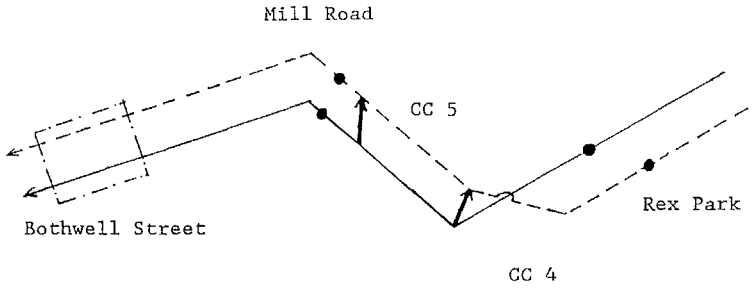
TABLE 10.22 Nett Runoff Volumes for Rex Park and Mill Road Parallel Pipe Catchments

EVENT	COMPUTED RUNOFF VOLUME		NETT RUNOFF VOLUME	
	FOUL (m^3)	STORM RELIEF (m^3)	FOUL (m^3)	STORM RELIEF (m^3)
860130	12413.29	8000.68	7793.62	6148.93
860510	9005.05	9757.05	6540.01	8768.96
860610	7711.45	13476.67	5282.93	12503.22
860612	4086.20	3380.25	1803.76	2465.36
860617	13354.62	47065.47	11072.18	46150.58
860806	4257.99	4205.02	2782.01	3576.79
860813	3275.82	3727.01	2241.11	3312.26
860815	7225.25	7592.36	4489.36	6128.60
860816	6123.00	10563.77	4163.14	9778.18
860902	5946.60	14711.52	4199.77	14011.32
861019	5741.73	4670.39	4262.71	4077.54
861019(B)	3769.39	3837.11	2357.32	3271.09

TABLE 10.23 Predicted and Nett Runoff Volumes for Bothwell Street Catchment

EVENT	NETT RUNOFF VOLUME			RATIO OF FOUL TO STORM RELIEF FLOWS
	FOUL (m^3)	STORM RELIEF (m^3)	COMBINED (m^3)	
860130	7793.62	6148.93	13942.55	1.27
860510	6540.01	8768.96	15308.97	0.75
860610	5282.93	12503.22	17786.15	0.42
860612	1803.76	2465.36	4269.12	0.73
860617	11072.18	46150.58	57222.76	0.24
860806	2782.01	3576.79	6358.80	0.78
860813	2241.11	3312.26	5553.37	0.68
860815	4489.36	6128.60	10617.96	0.73
860816	4163.14	9778.18	13941.32	0.43
860902	4199.77	14011.32	18211.09	0.30
861019	4262.71	4077.54	8340.25	1.05
			AVERAGE = 0.675 (67.5%)	

TABLE 10.24 Combined Runoff Volume and Ratio of flows in Parallel Pipes for Bothwell Street Catchment



Foul Pipes
Storm Relief Pipes
Flow Monitoring Locations
Overflow Chamber
Cross-Connections

EVENT	BOTHWELL STREET	MILL ROAD		REX PARK	
	COMBINED MASS FLOW (m^3)	COMBINED MASS FLOW (m^3)	% OF FLOW RELATED TO BOTHWELL (%)	COMBINED MASS FLOW (m^3)	% OF FLOW RELATED TO BOTHWELL (%)
860510	15308.97	/	/	14739.67	96.3
860617	57222.76	/	/	27987.32	48.9
860806	6358.80	3558.32	56.0	/	/
860813	5553.37	4077.42	73.4	/	/
860816	13941.32	8526.70	61.2	/	/
860902	18211.09	9376.75	51.5	/	/
		Average = 60.5		Average = 72.6	

TABLE 10.25 Combined Flow Volumes and Related Flow Percentage for the Three Catchments

EVENT DATE	RAIN-FALL DEPTH (mm)	REX PARK		MILL ROAD		BOTHWELL STREET	
		RUNOFF (m^3)	RUNVOL (mm)	RUNOFF (m^3)	RUNVOL (mm)	RUNOFF (m^3)	RUNVOL (mm)
860130	8.2	/	/	/	/	13942.55	3.7275
860510	14.4	14739.67	8.4181	/	/	15308.97	4.0929
860512	5.7	2565.22	1.4650	/	/	/	/
860517	11.2	6596.50	3.7674	/	/	/	/
860610	16.9	/	/	/	/	17786.15	4.7551
860612	6.2	/	/	/	/	4269.12	1.1414
860617	32.6	27987.32	15.9840	/	/	57222.76	15.2985
860806	9.1	/	/	3558.32	1.5880	6358.80	1.7000
860813	5.9	/	/	4077.42	1.8196	5553.37	1.4847
860815	14.2	/	/	/	/	10617.96	2.8387
860816	11.3	/	/	8526.70	3.8052	13941.32	3.7272
860902	18.1	/	/	9376.75	4.1846	18211.09	4.8687
861019	8.1	/	/	/	/	8340.25	2.2298
861019 (B)	5.0	/	/	/	/	5628.41	1.5048

TABLE 10.26 List of Events and the Corresponding Combined Runoff Volume and RUNVOL for the Three Catchments

LOCATION	RUNOFF COEFFICIENT (RC)		DEPRESSION STORAGE (DEPSTOG)	
	OBSERVED DATA	PARALLEL PIPE MODEL	OBSERVED DATA (mm)	PARALLEL PIPE MODEL (mm)
REX PARK	0.80	0.54	2.84	2.23
MILL ROAD	0.40	0.33	2.99	2.71
BOTHWELL STREET	0.62	0.47	3.36	4.04
Average	0.61	0.45	3.06	2.99

TABLE 10.27 Comparison of Runoff Coefficient (RC) and Depression Storage (DEPSTOG) Values between those Derived Using Observed Data and the Parallel Pipe Model Results

LOCATION	EVENT DATE	FOUL			STORM RELIEF		
		OBSERVED PEAK LEVEL (m)	PREDICTED PEAK LEVEL (m)	% DIFF. (%)	OBSERVED PEAK LEVEL (m)	PREDICTED PEAK LEVEL (m)	% DIFF. (%)
REX PARK	860517	0.545	0.5716	-4.9	0.474	0.4809	-1.5
MILL ROAD	860813	0.333	0.3379	-1.5	0.525	0.5431	-3.4
BOTHWELL STREET	860617	0.700 0.742 0.612	0.7114 0.7638 0.5986	-1.6 -2.9 +2.2	2.023 1.442 1.142	2.0971 1.5101 1.1883	-3.7 -4.7 -4.1

TABLE 10.28 Comparison of Observed and Predicted Peak Levels in Parallel Pipes for Three Catchments

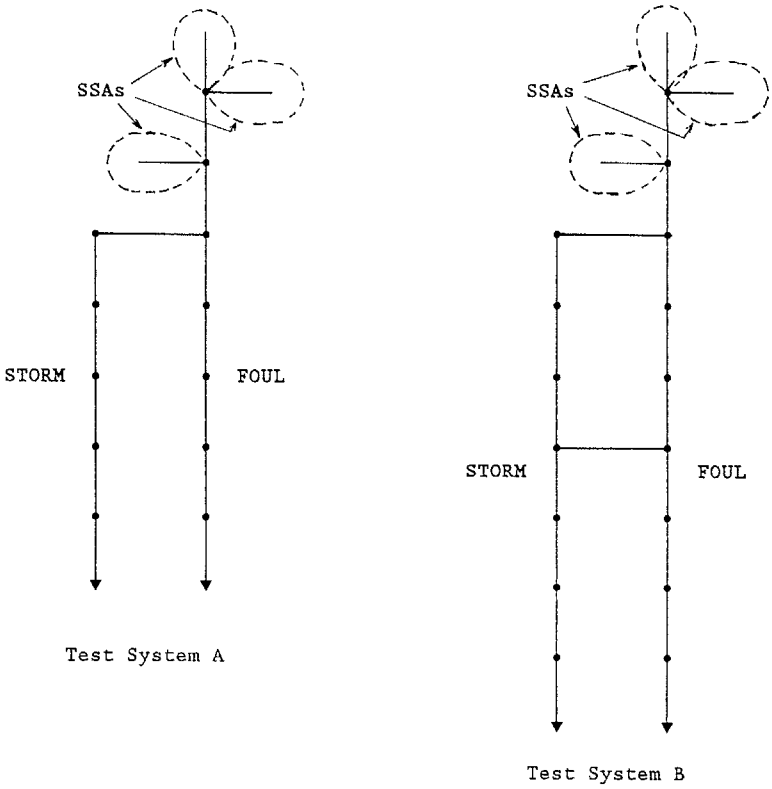


FIGURE 10.1 SCHEMATIC SKETCH OF 'IDEALISED' TEST SYSTEMS

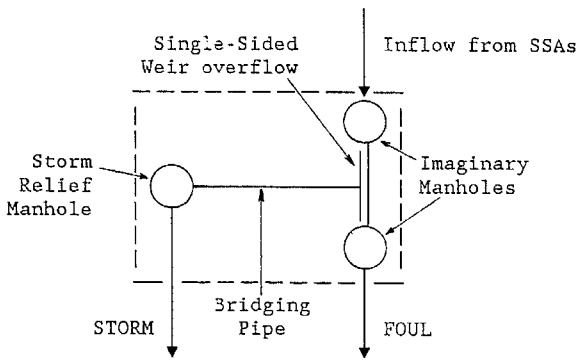


FIGURE 10.2 SCHEMATIC SKETCH OF CROSS-CONNECTION IN TEST SYSTEMS A AND B

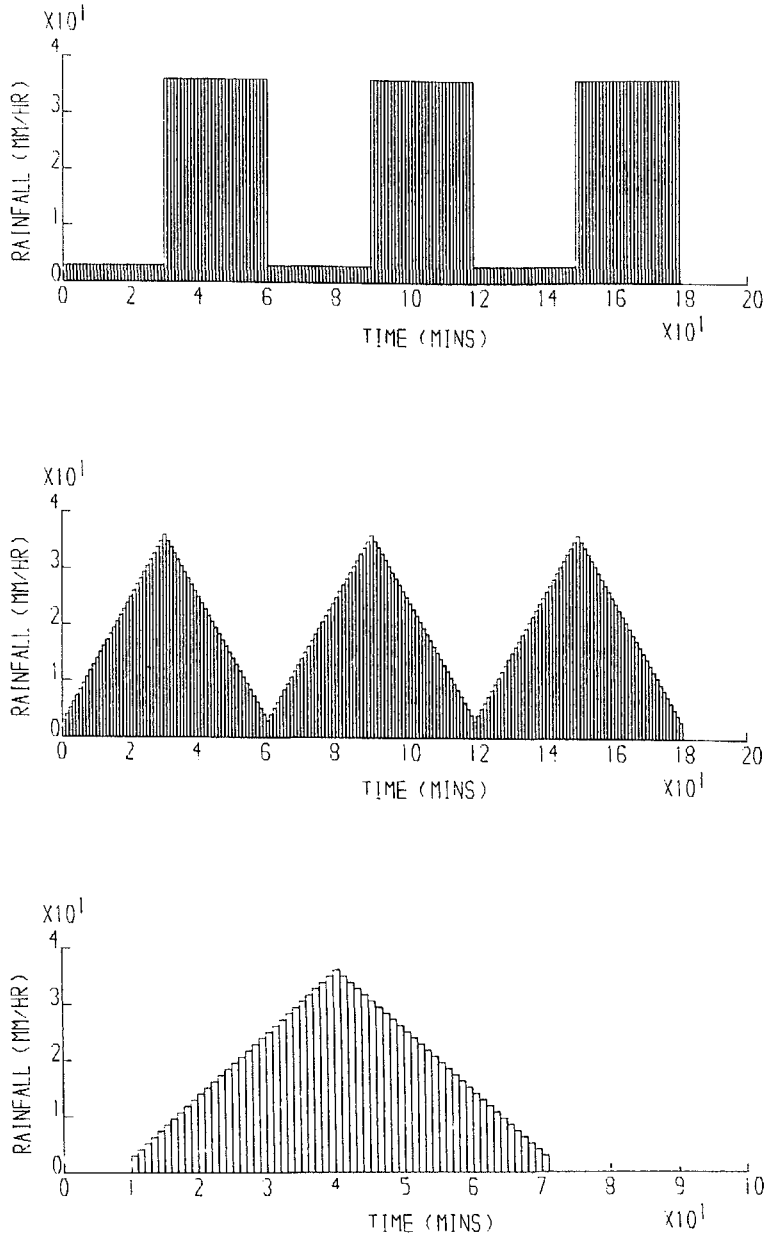
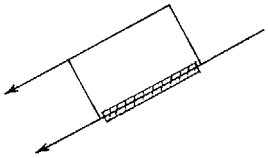
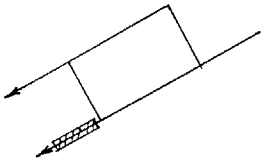


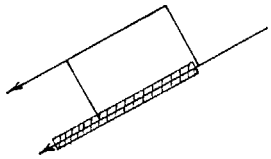
FIGURE 10.3 The Three Artifical Test Rainfall Patterns



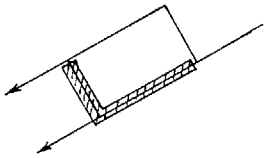
(i) Surcharge at Upper Leg of Foul
(occur with test systems B1, B4)



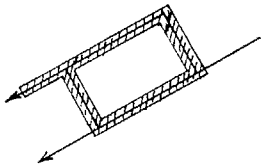
(ii) Surcharge at Lower Leg of Foul
(occur with test systems B2, B3)



(iii) Surcharge along the Entire Foul Leg
(occurs with test system B1)



(iv) Surcharge at the Upper Foul Leg and Second Bridging Pipe
(occurs with test system B3)



(v) Surcharge at Upper Foul Leg, Bridging Pipes and the Entire Storm Relief Leg
(occurs with test system B5)

FIGURE 10.4 PROPOSED SURCHARGE SUB-SYSTEM SET-UPS FOR COMPONENT TESTING

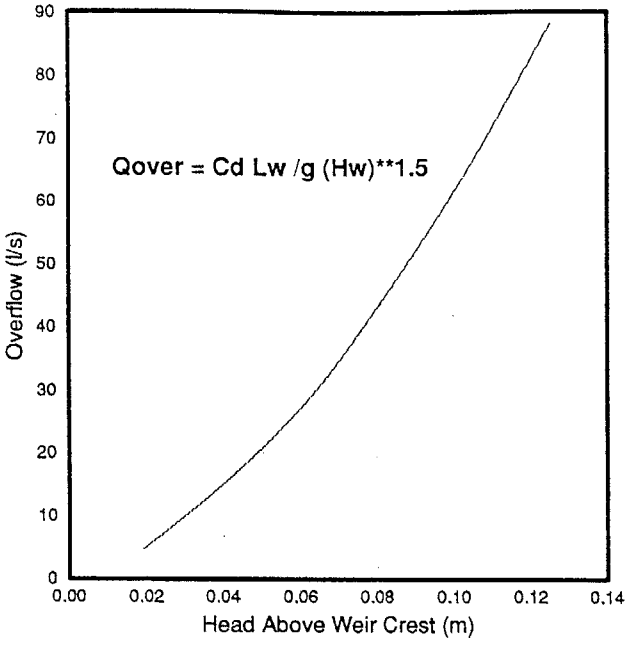


FIGURE 10.5 OVERFLOW AGAINST WEIR HEAD FOR WEIR TYPE 1

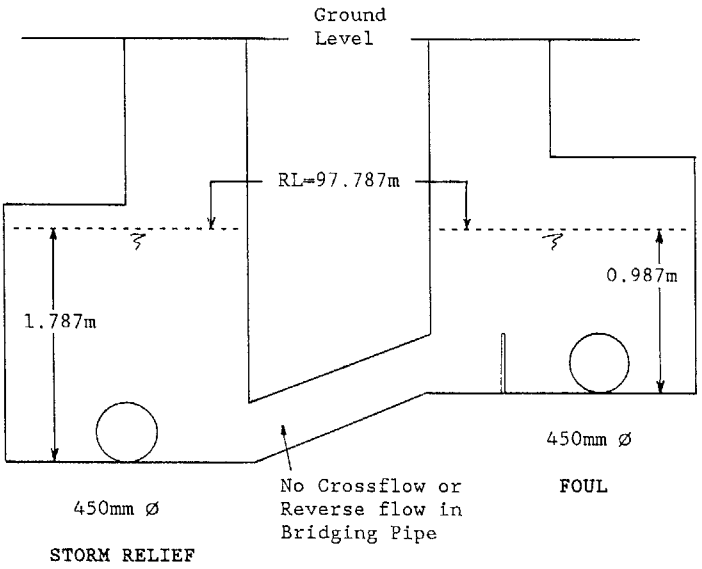


FIGURE 10.6 CROSS SECTIONAL SKETCH SHOWING BALANCE OF HEADS IN CROSS-CONNECTION MANHOLES

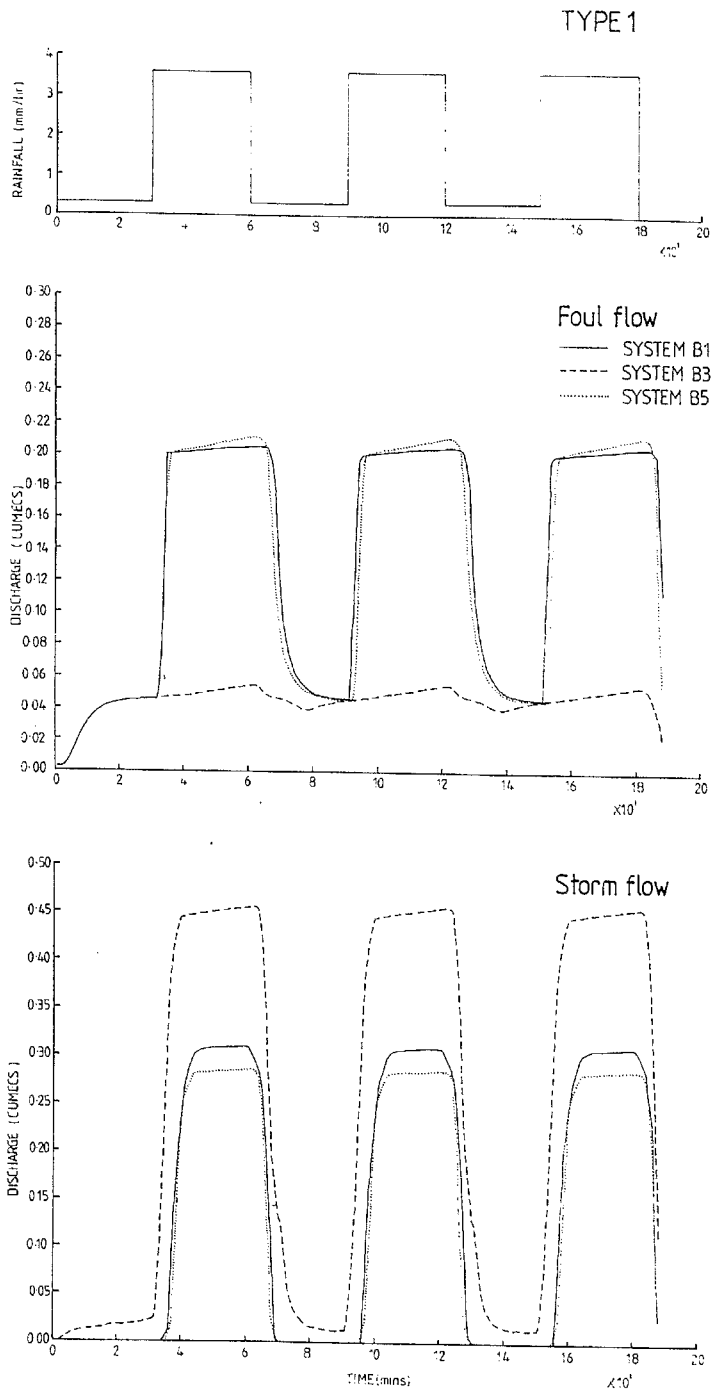


FIGURE 10.7 Simulation of Flows for Parallel Pipes for Test Systems B1, B3 and B5 with Test Rainfall Type 1

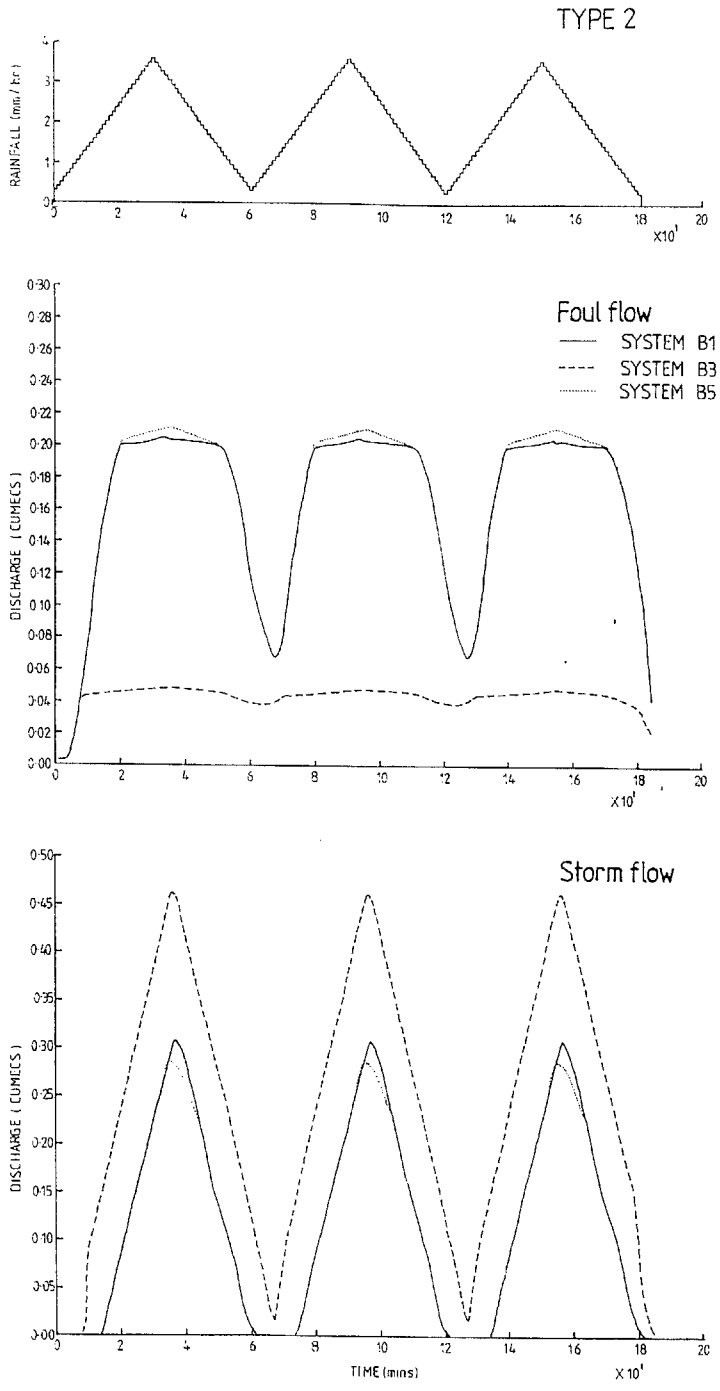


FIGURE 10.8 Simulation of Flows for Parallel Pipes for Test Systems B1, B3 and B5 with Test Rainfall Type 2

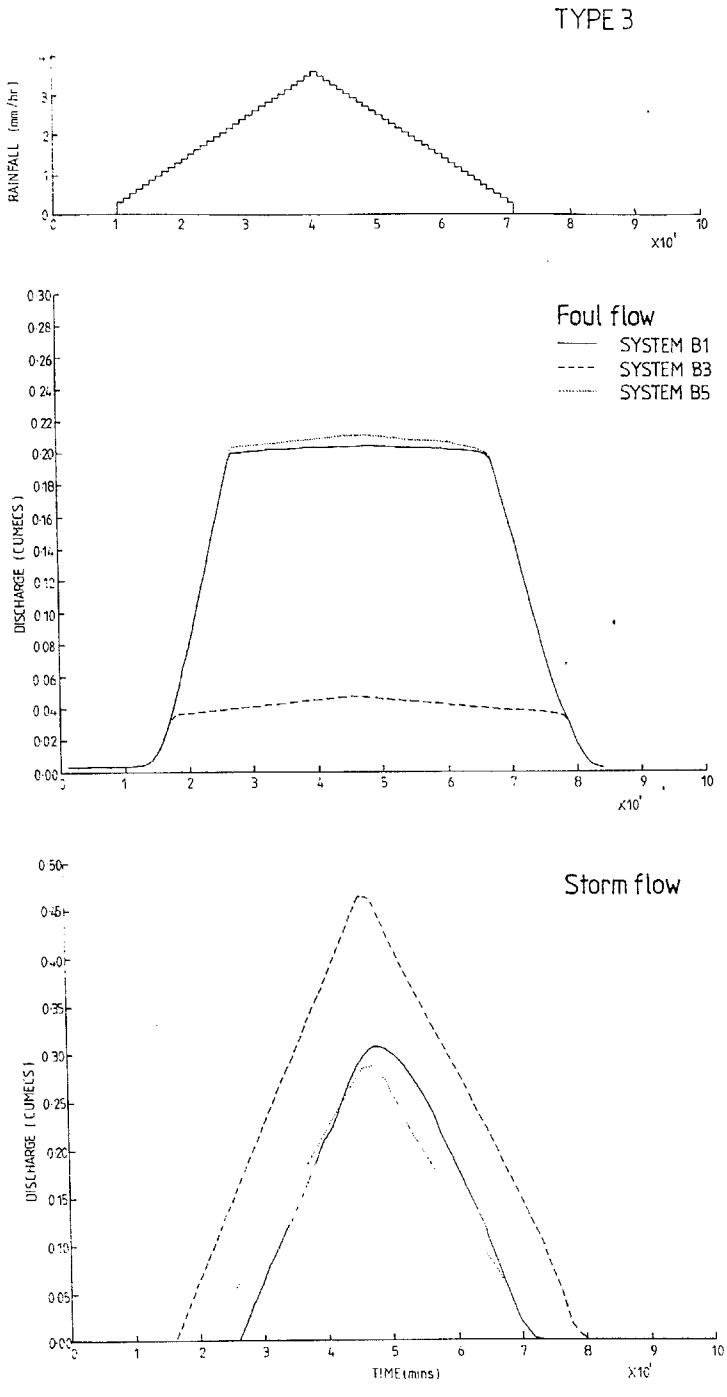


FIGURE 10.9 Simulation of Flows for Parallel Pipes for Test Systems B1, B3 and B5 with Test Rainfall Type 3

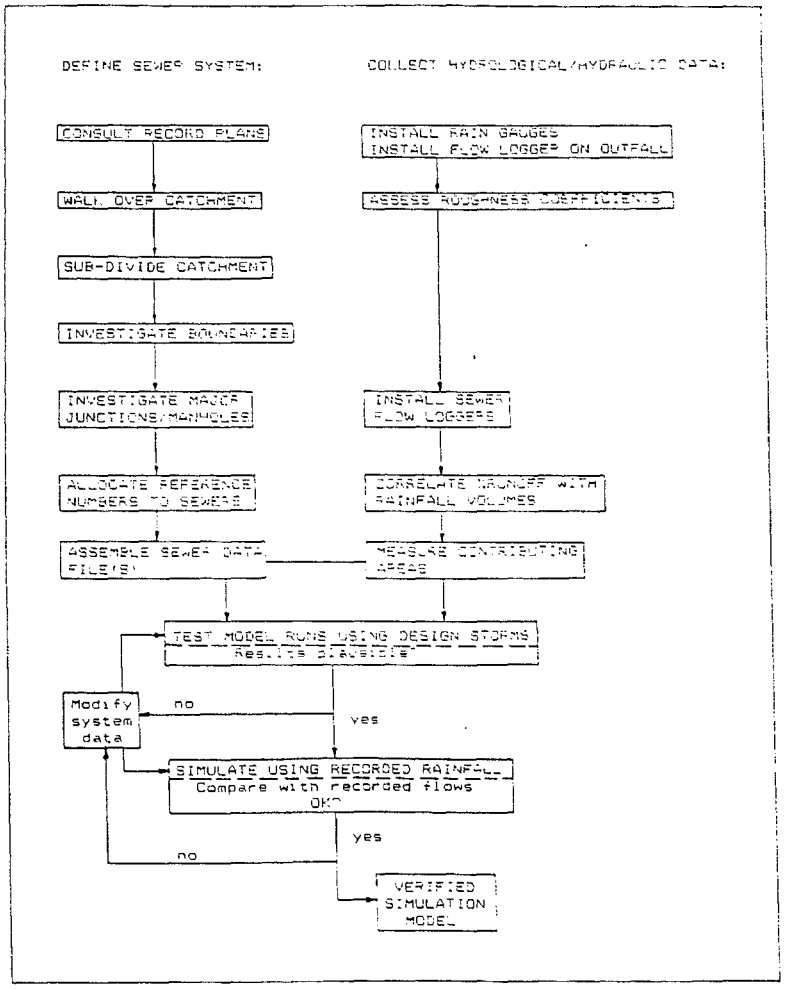


FIGURE 10.10 Setting Up a Sewer Flow Simulation Model

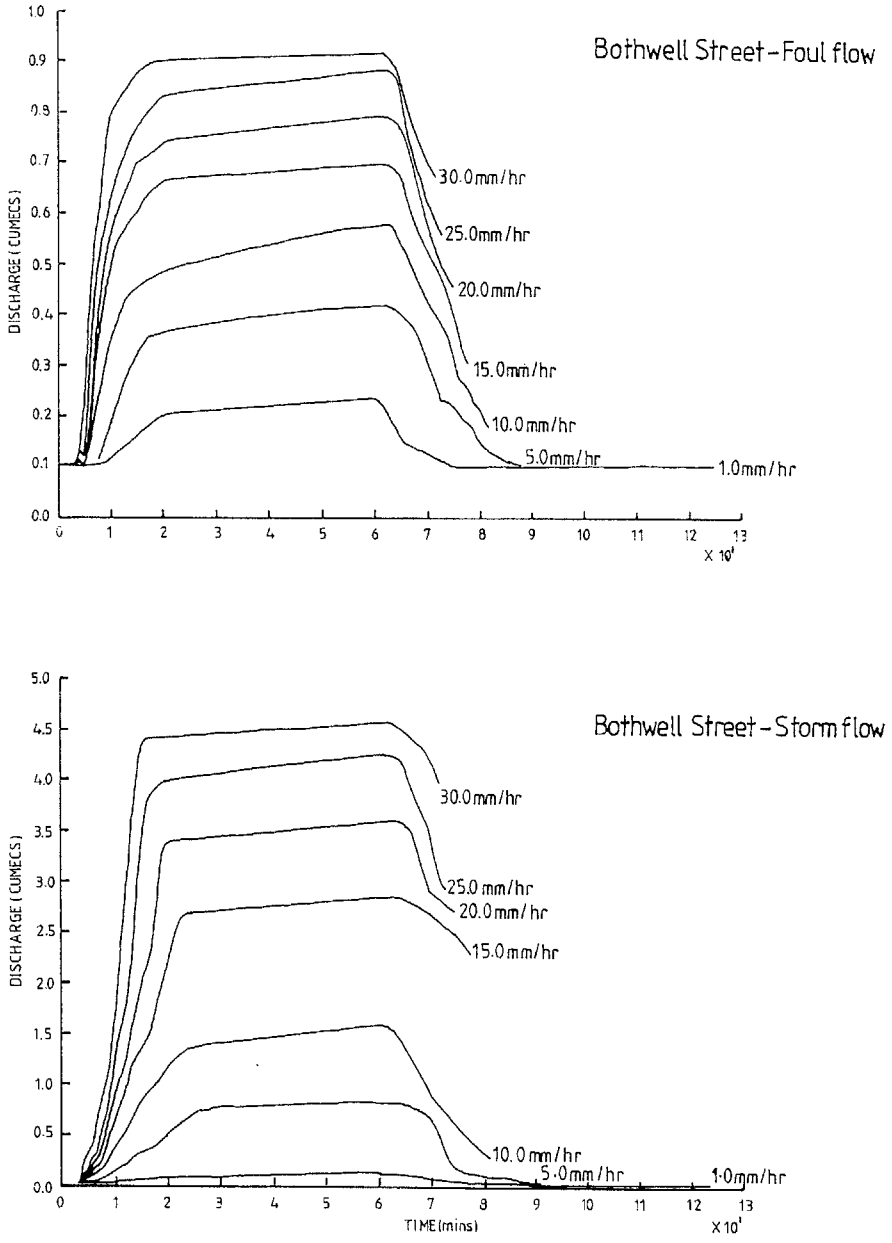


FIGURE 10.11 Response of Flows in Parallel Pipes at Bothwell Street under Various Uniform Rainfalls

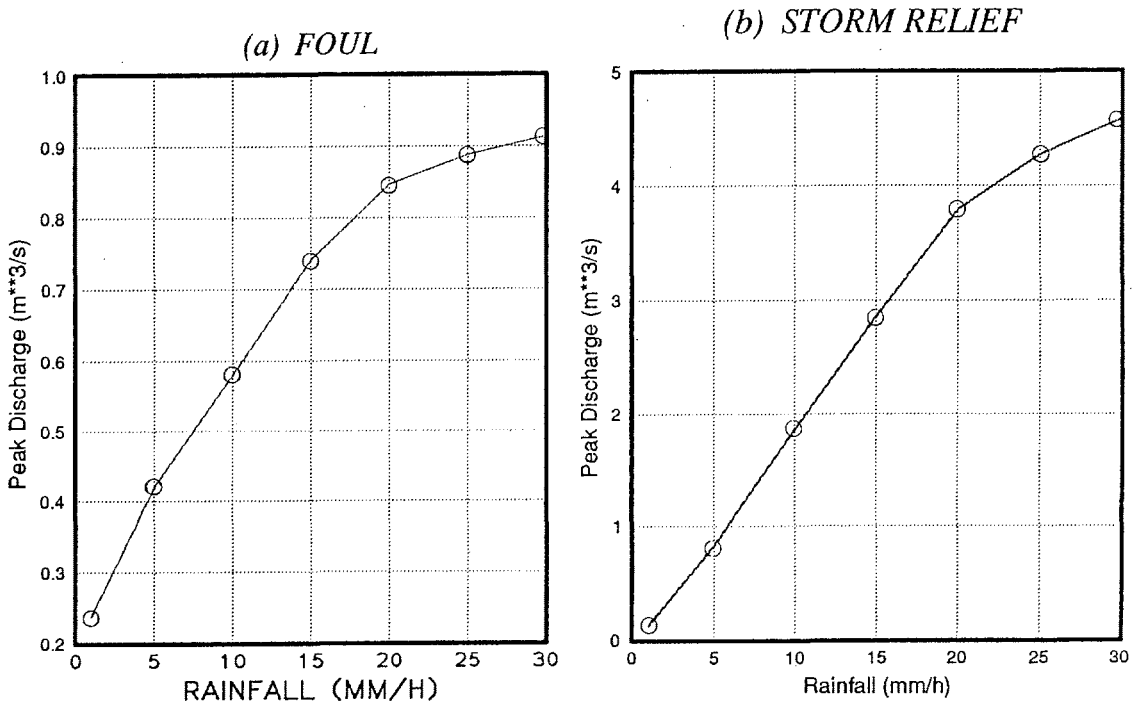
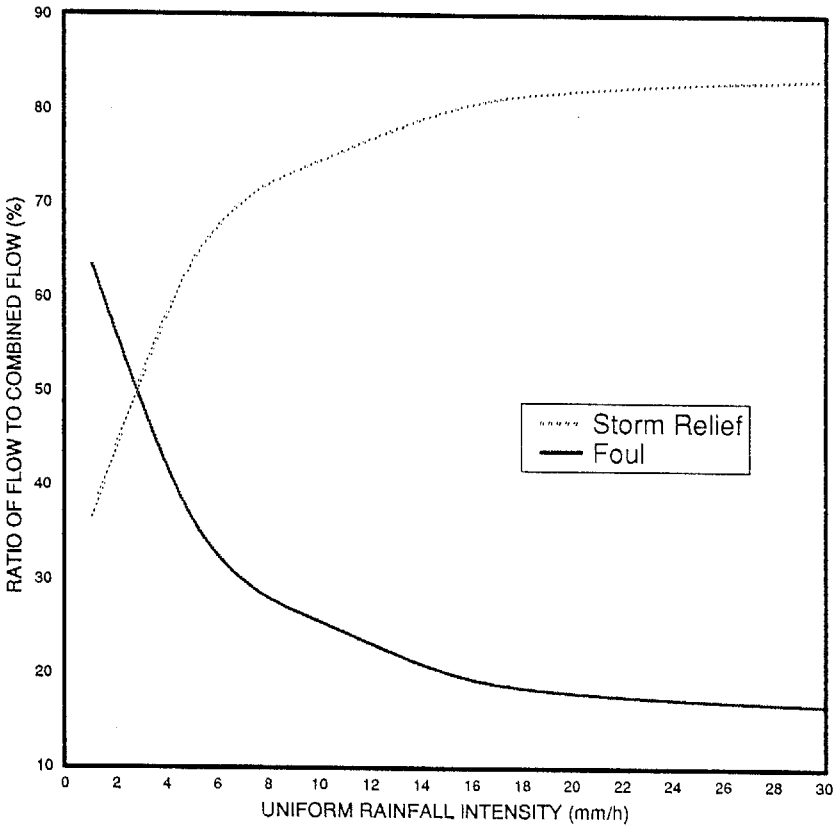


FIGURE 10.12 Peak Discharges against Uniform Rainfalls for Parallel Pipes at Bothwell Street

FIGURE 10.13 RATIO OF FLOW TO COMBINED FLOW AGAINST UNIFORM RAINFALL



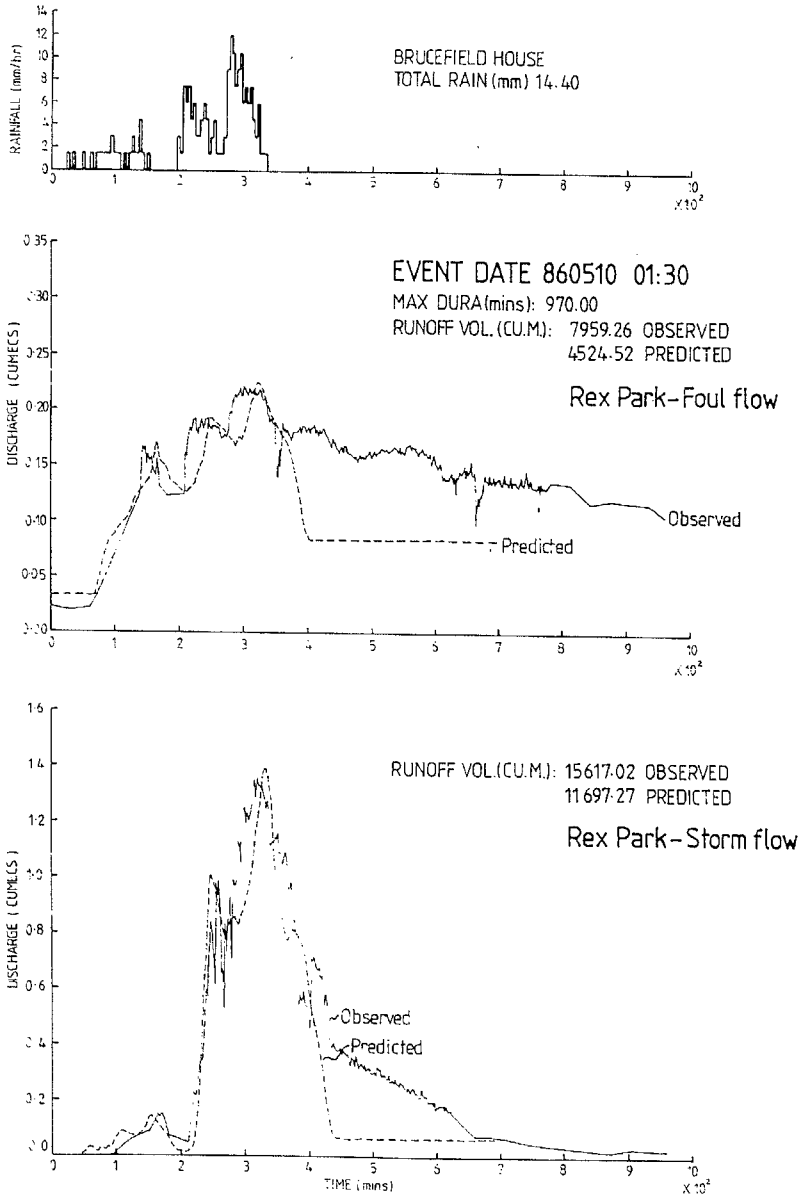


FIGURE 10.14 Comparison of Observed and Predicted (DUPPERS) Flows for Parallel Pipes at the Outfalls of Rex Park Catchment

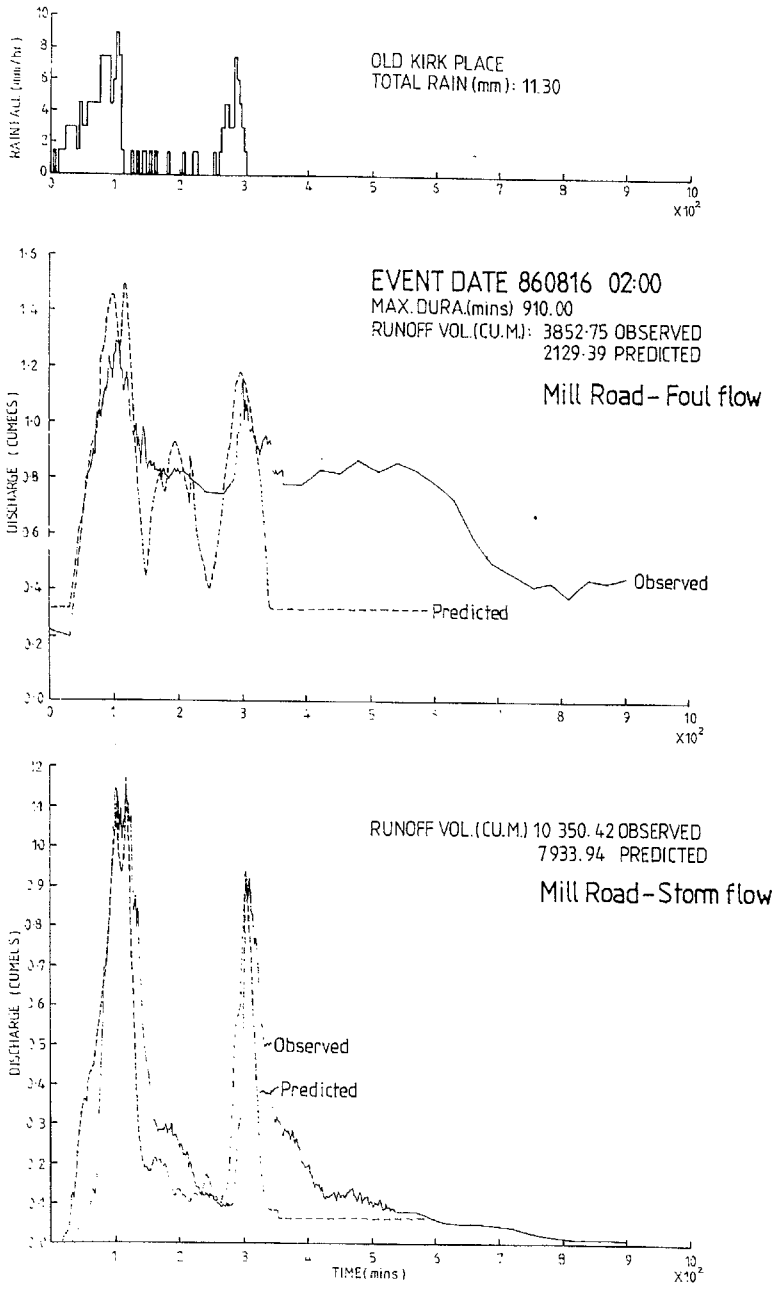


FIGURE 10.15 Comparison of Observed and Predicted (DUPPERS) Flows for Parallel Pipes at the Outfalls of Mill Road Catchment

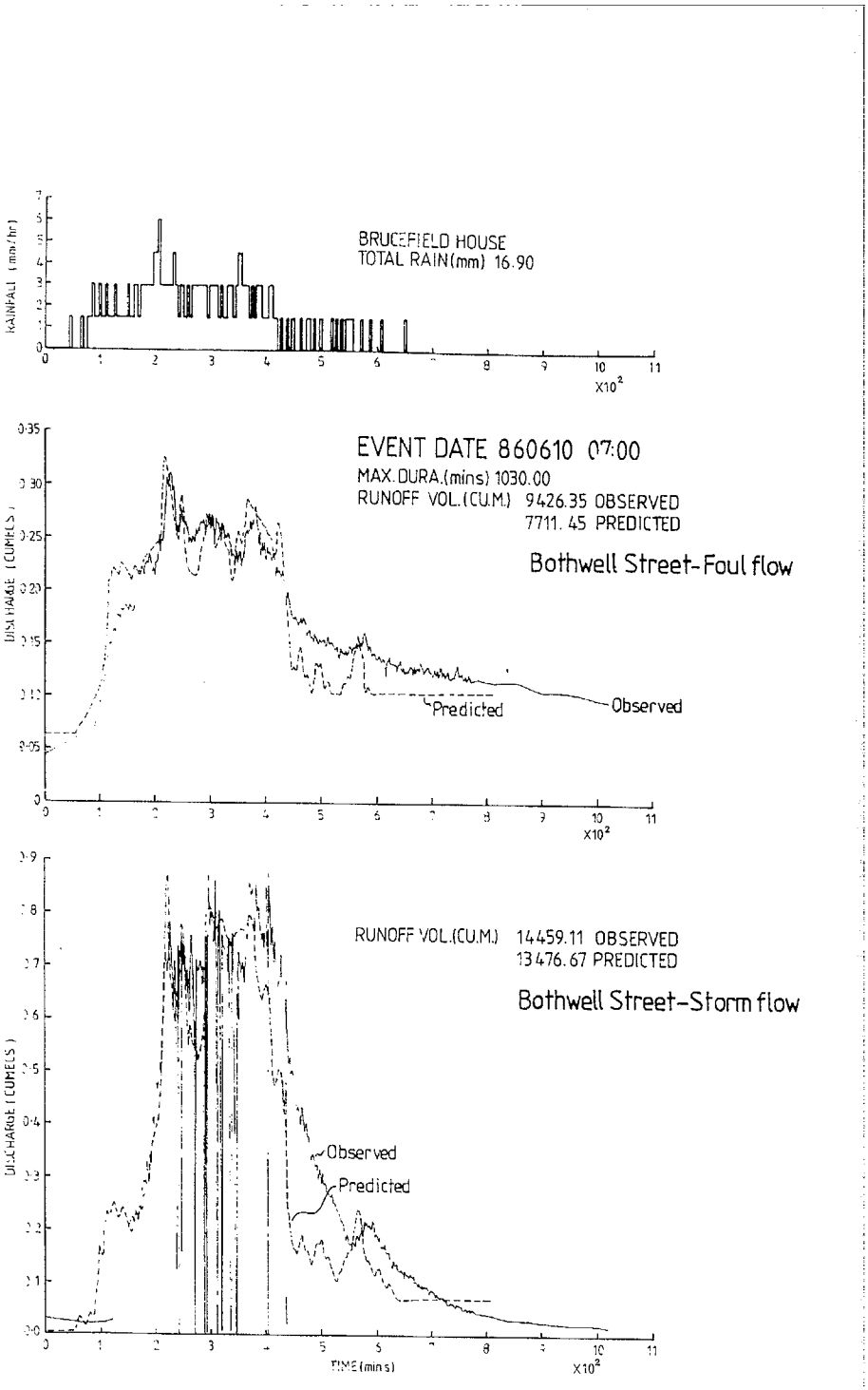


FIGURE 10.16 Comparison of Observed and Predicted (DUPPERS) Flows for Parallel Pipes at the Outfalls of Bothwell St Catchment

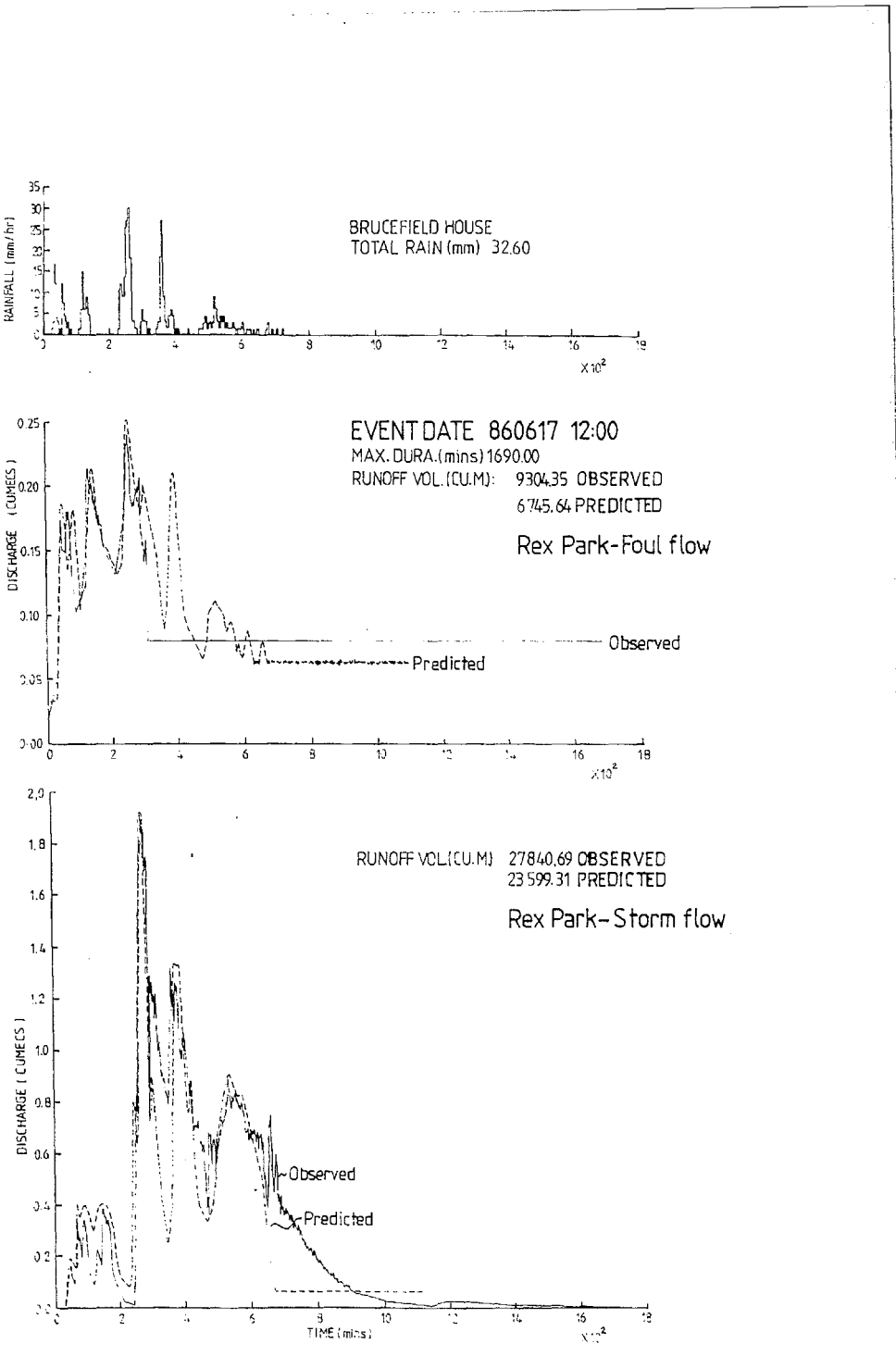


FIGURE 10.17 Verification of Model by Comparing the Observed Data and DUPPERS Simulated Outputs for Parallel Pipes at Rex Park

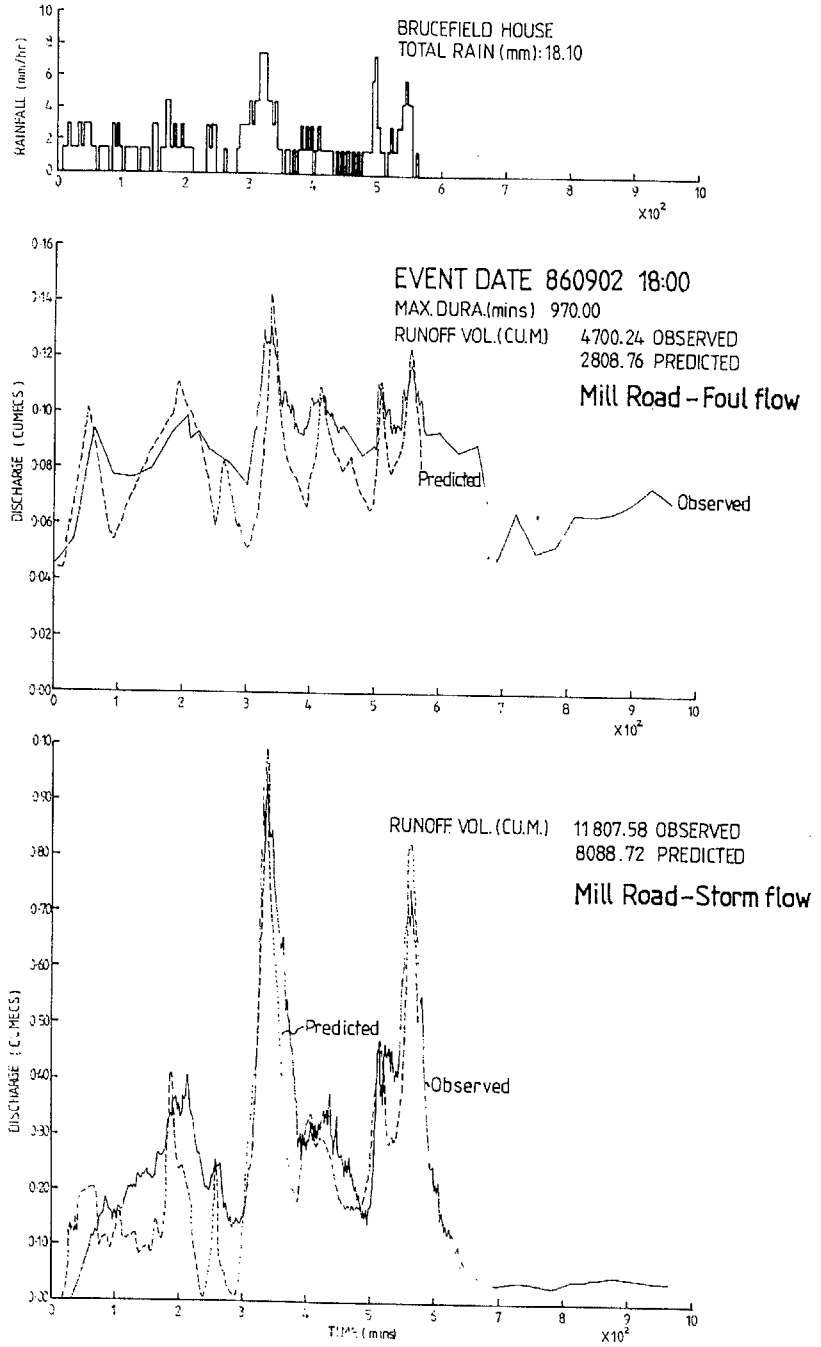


FIGURE 10.18 Verification of Model by Comparing the Observed Data and DUPPERS Simulated Outputs for Parallel Pipes at Mill Road

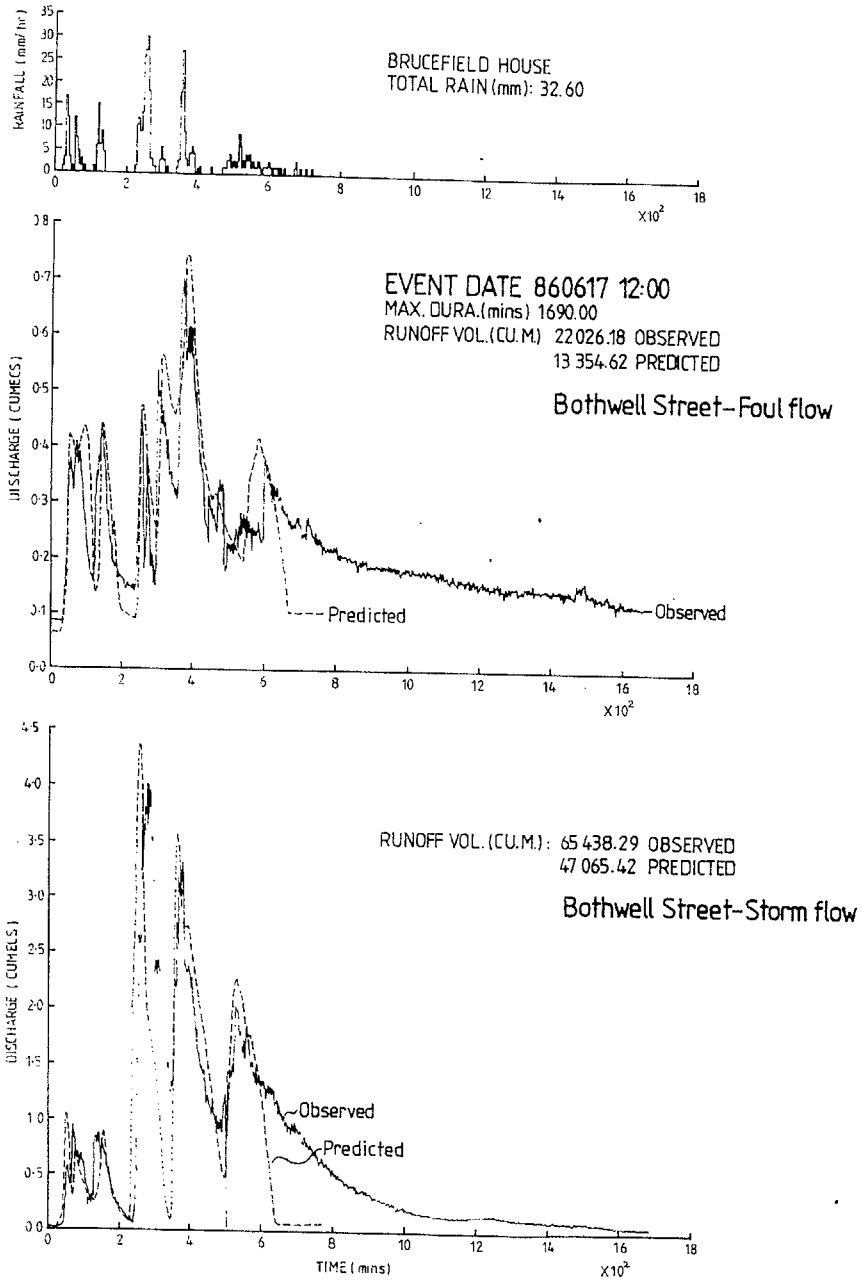


FIGURE 10.19 Verification of Model by Comparing the Observed Data and DUPPERS Simulated Outputs for Parallel Pipes at Bothwell Street Chamber

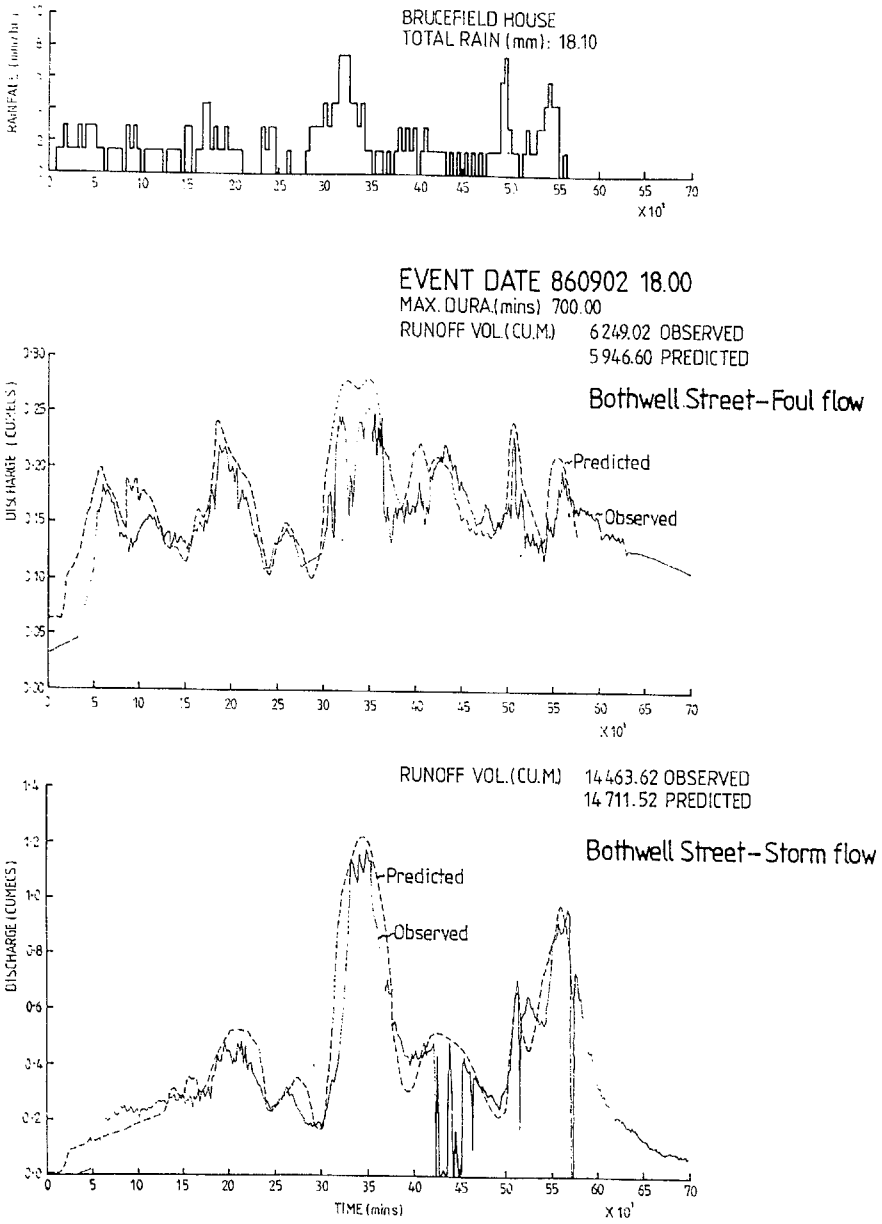


FIGURE 10.20 Verification of Model by Comparing the Observed Data and DUPPERS Simulated Outputs for Parallel Pipes at Bothwell Street Chamber

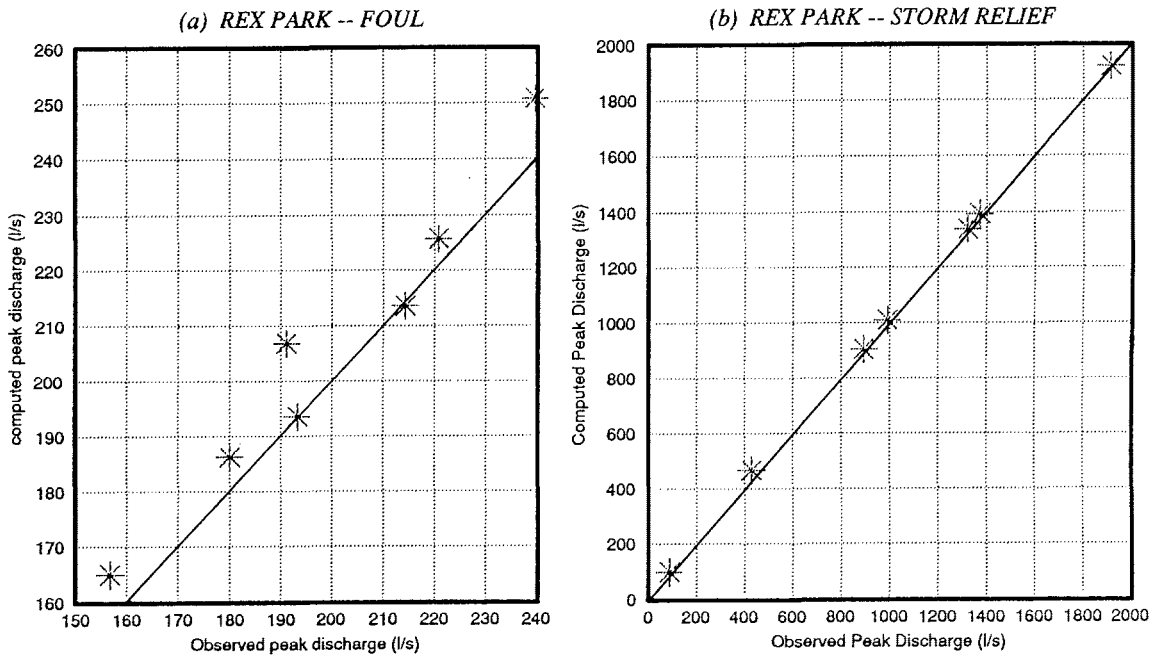
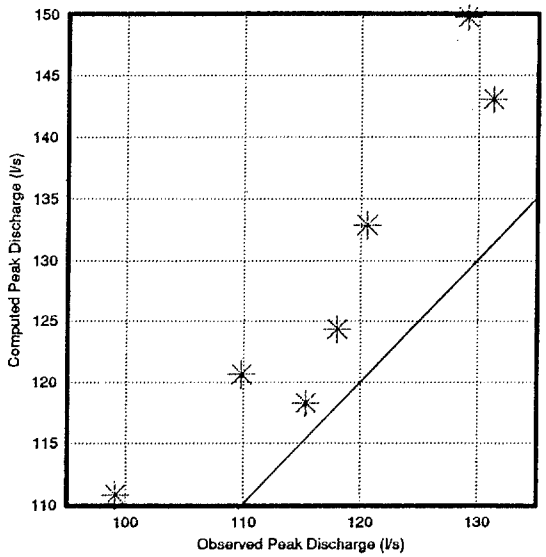


FIGURE 10.21 Computed against Observed Peak Discharges for Parallel Pipes at Rex Park

(a) MILL ROAD -- FOUL



(b) MILL ROAD -- STORM RELIEF

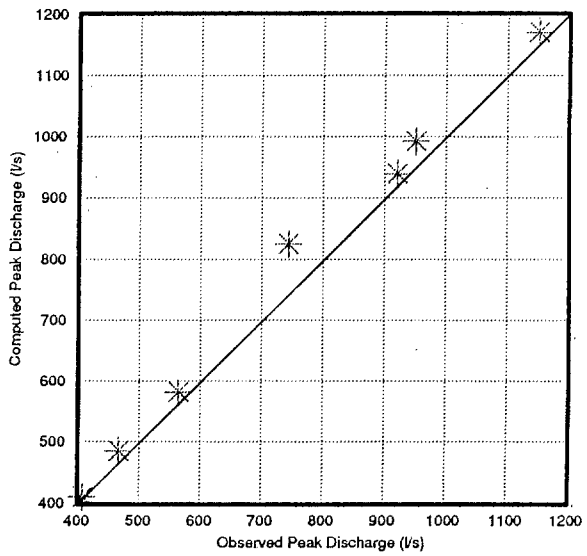


FIGURE 10.22 Computed against Observed Peak Discharges for Parallel Pipes at Mill Road

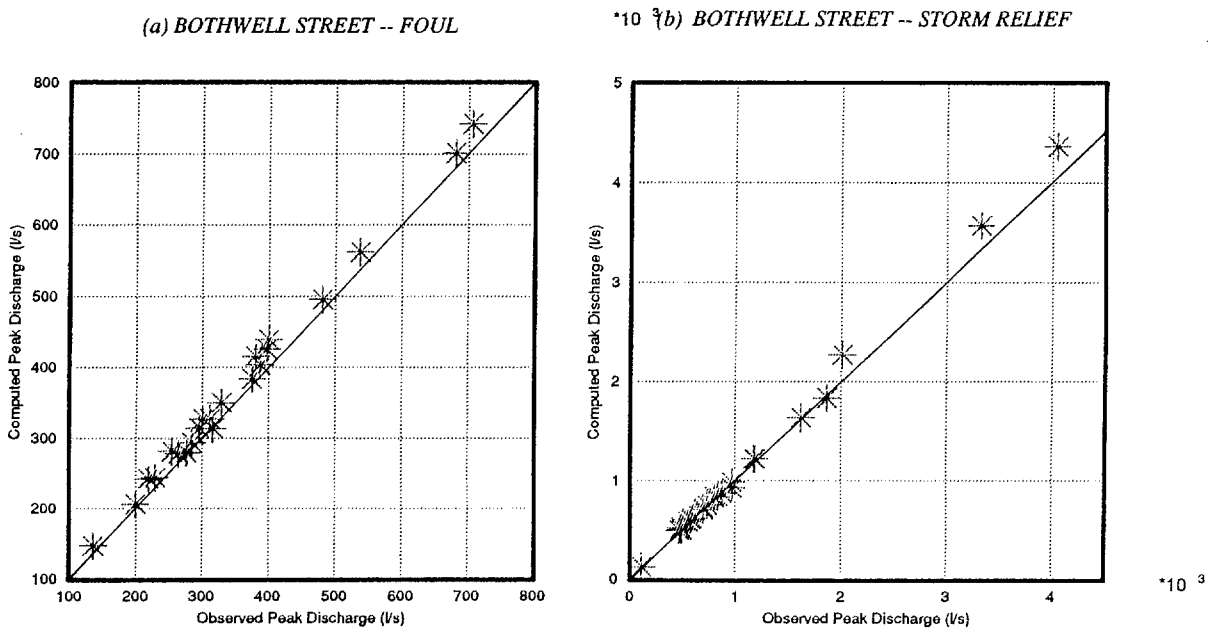
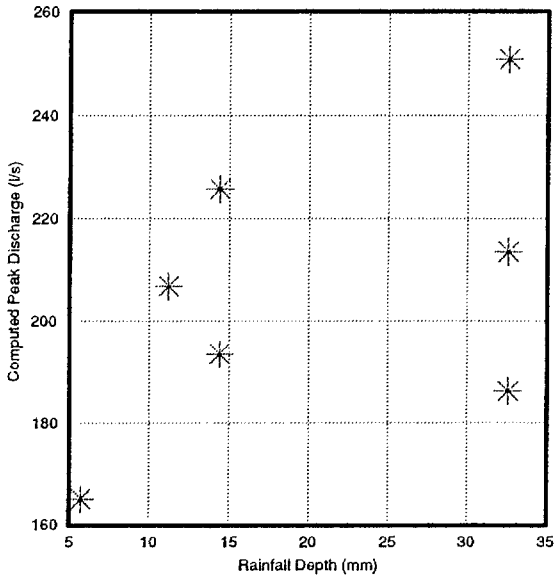


FIGURE 10.23 Computed against Observed Peak Discharges for Parallel Pipes at Bothwell Street

(a) REX PARK -- FOUL



(b) REX PARK -- STORM RELIEF

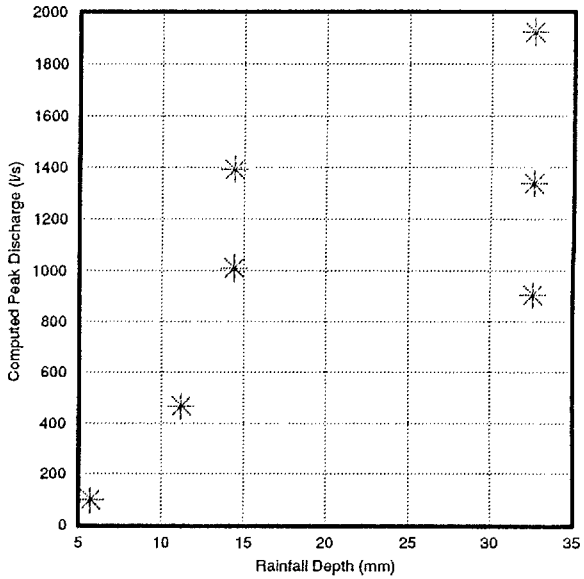
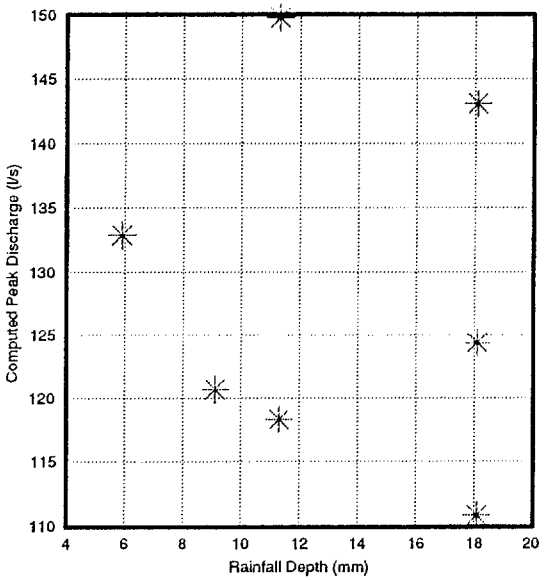


FIGURE 10.24 Computed Peak Discharges against Rainfall Depths for Parallel Pipes at Rex Park

(a) MILL ROAD -- FOUL



(b) MILL ROAD -- STORM RELIEF

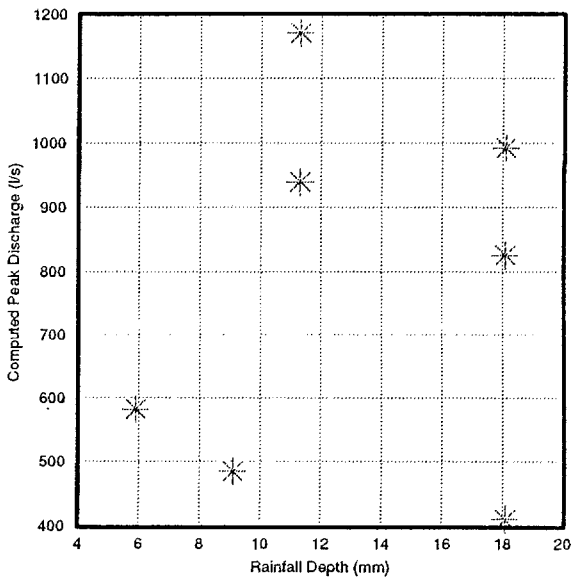


FIGURE 10.25 Computed Peak Discharges against Rainfall Depths for Parallel Pipes at Mill Road

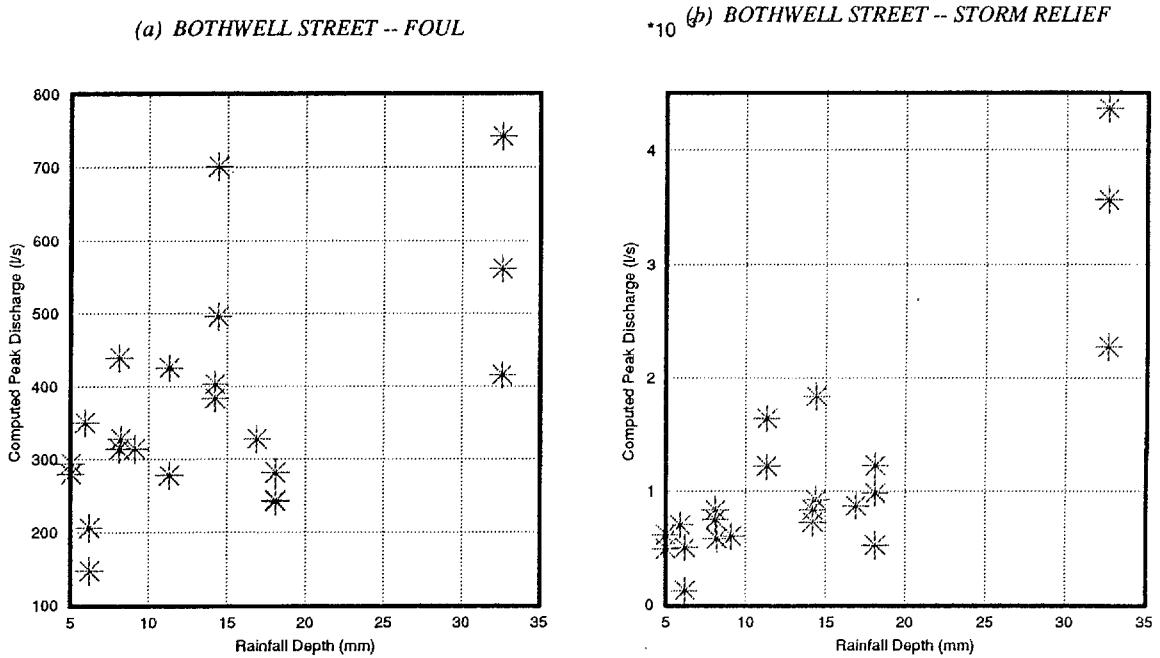


FIGURE 10.26 Computed Peak Discharges against Rainfall Depths for Parallel Pipes at Bothwell Street

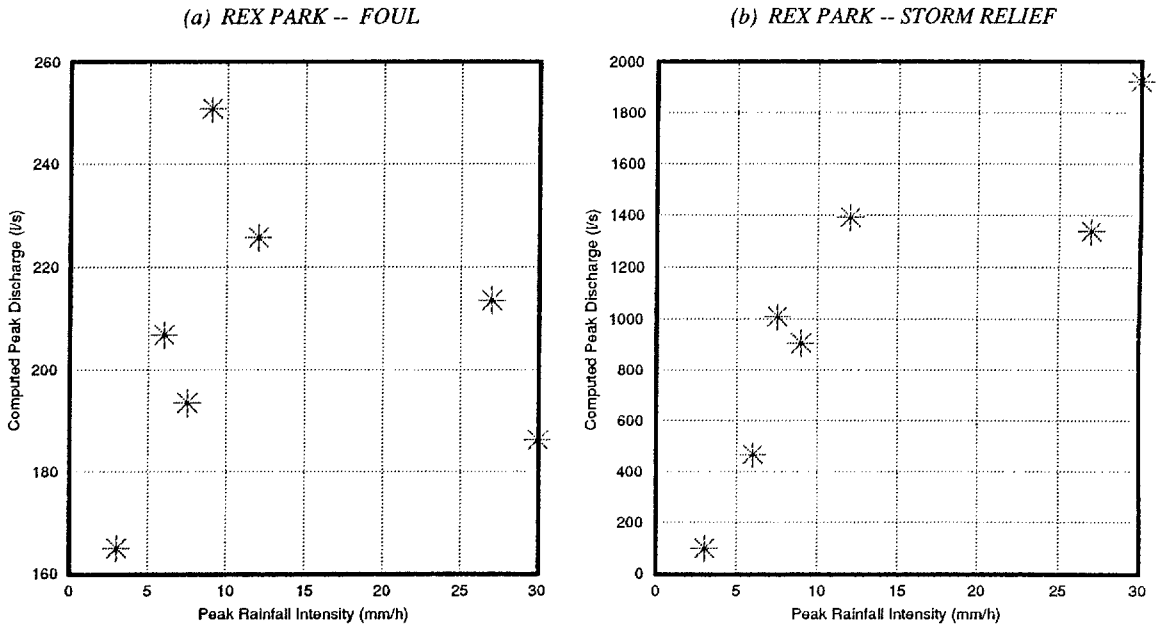


FIGURE 10.27 Computed Peak Discharges against Peak Rainfall Intensities for Parallel Pipes at Rex Park

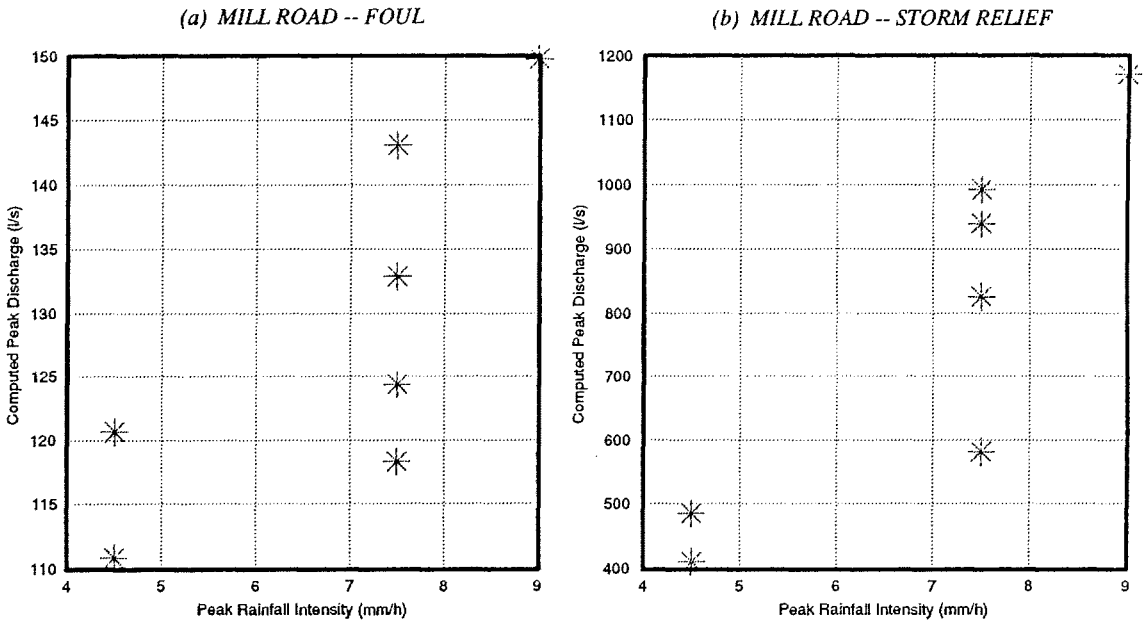


FIGURE 10.28 Computed Peak Discharges against Peak Rainfall Intensities for Parallel Pipes at Mill Road

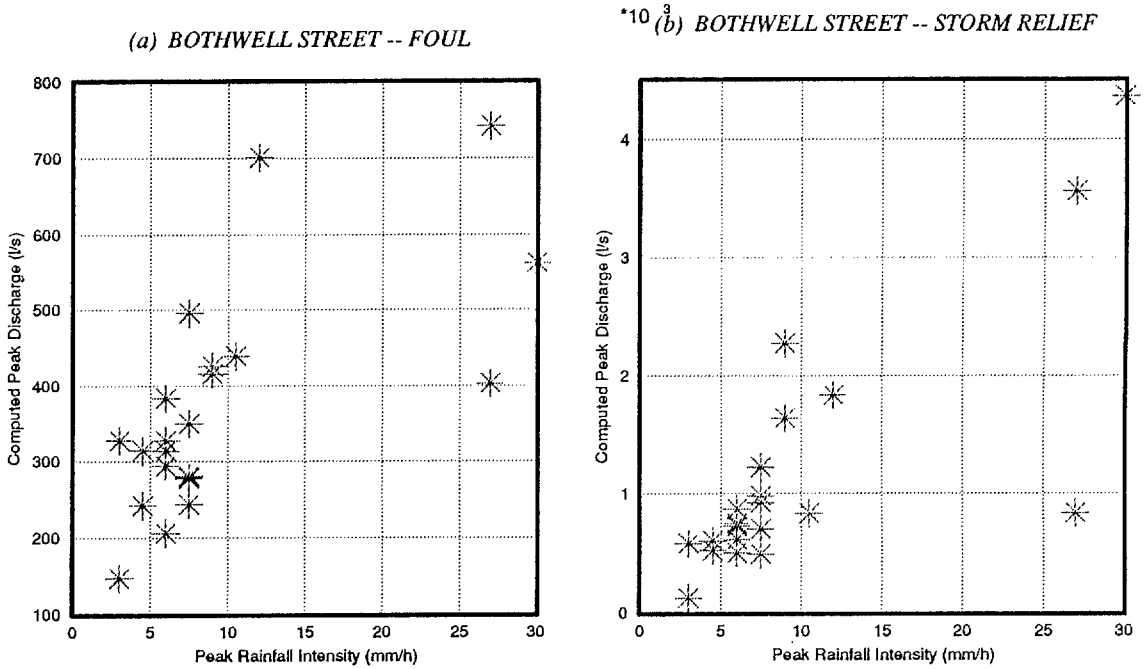
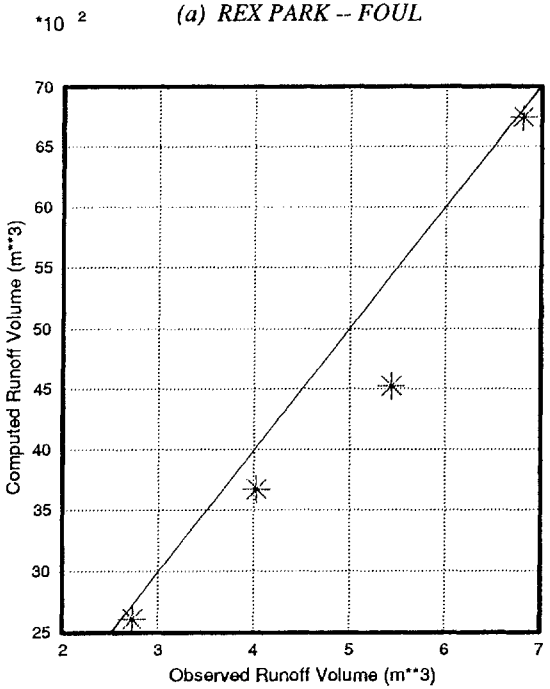


FIGURE 10.29 Computed Peak Discharges against Peak Rainfall Intensities for Parallel Pipes at Bothwell Street

(a) REX PARK -- FOUL



(b) REX PARK -- STORM RELIEF

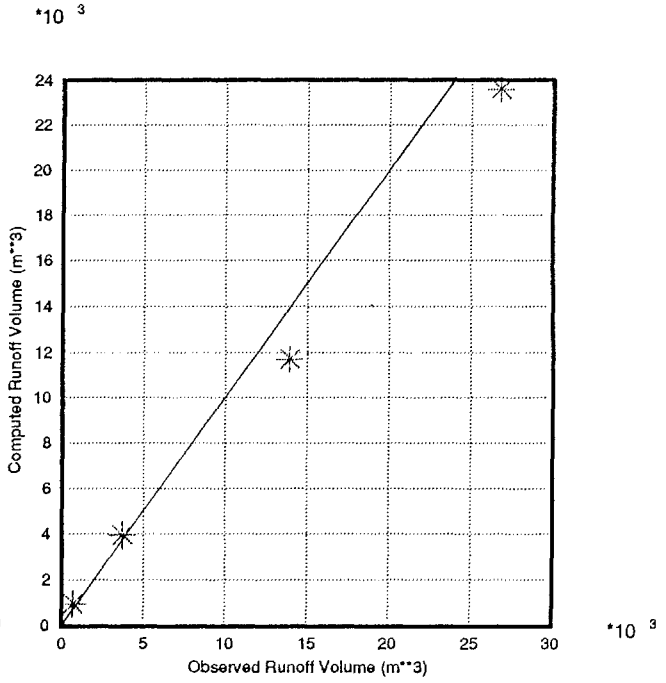
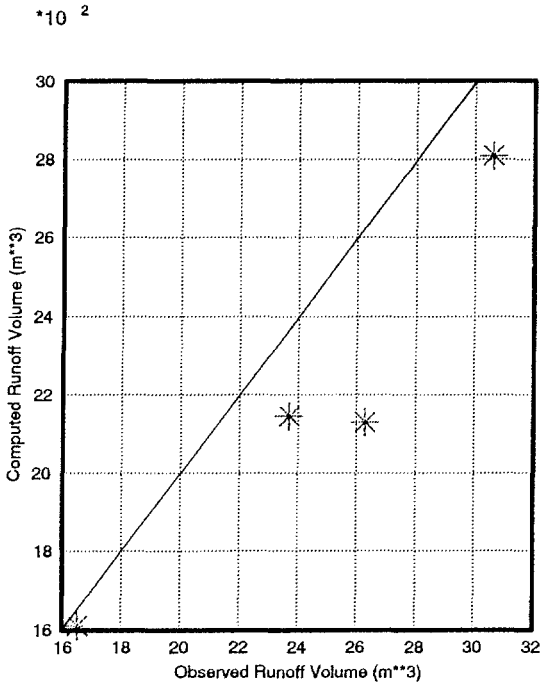


FIGURE 10.30 Computed Total Runoff Volumes against Total Observed Runoff Volumes for Parallel Pipes at Rex Park

(a) MILL ROAD -- FOUL



(b) MILL ROAD -- STORM RELIEF

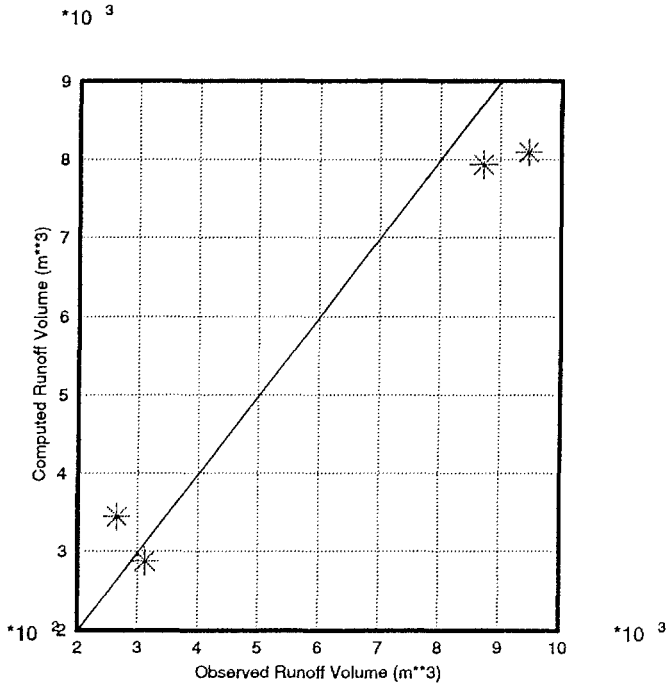
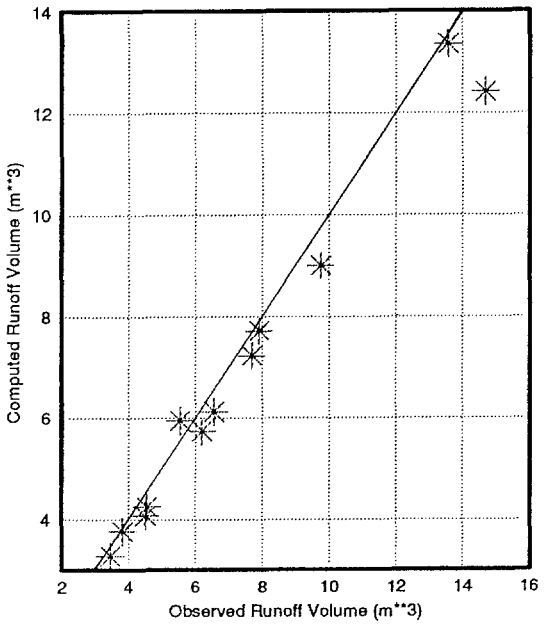


FIGURE 10.31 Computed Total Runoff Volumes against Total Observed Runoff Volumes for Parallel Pipes at Mill Road

(a) BOTHWELL STREET -- FOUL



(b) BOTHWELL STREET -- STORM RELIEF

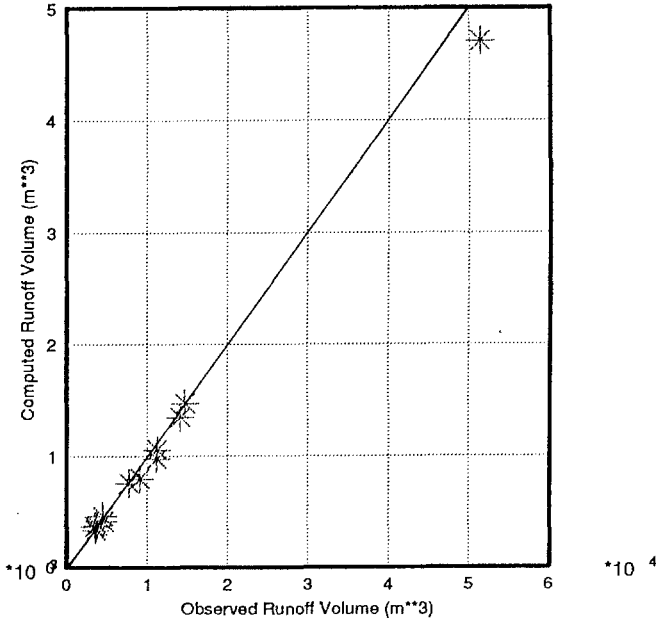


FIGURE 10.32 Computed Total Runoff Volumes against Total Observed Runoff Volumes for Parallel Pipes at Bothwell Street

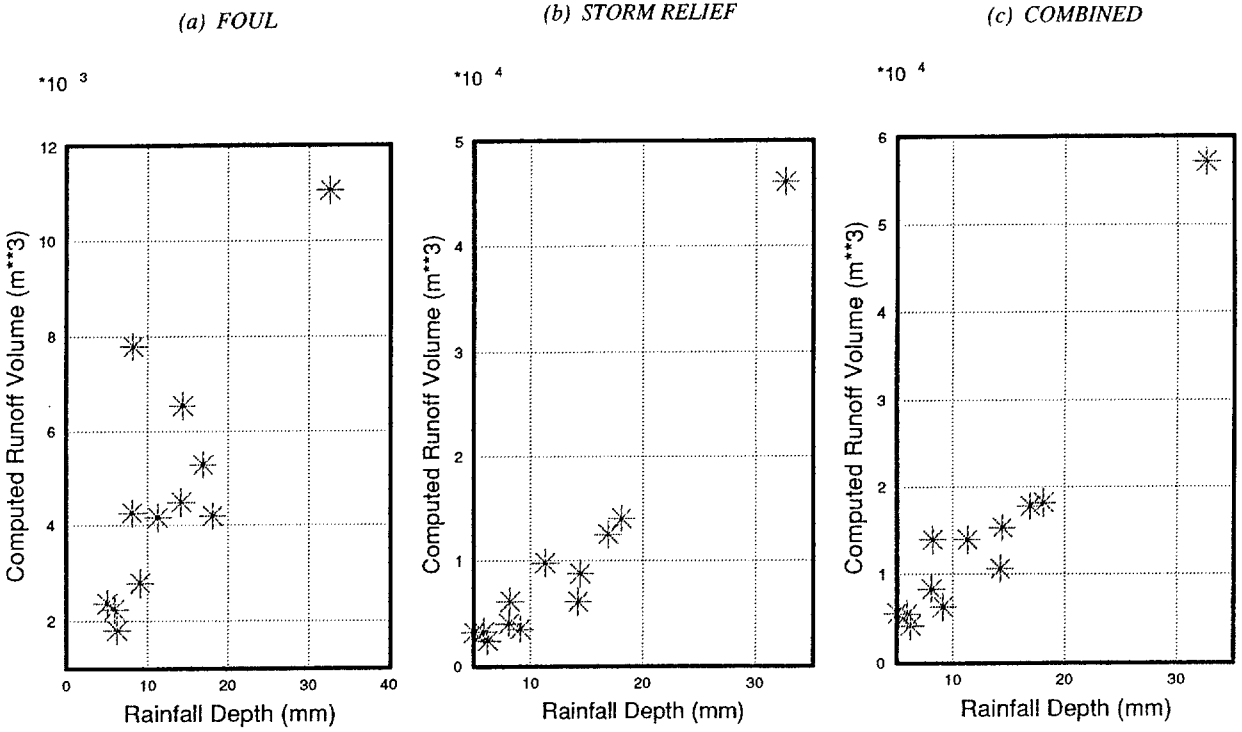


FIGURE 10.33 Computed Runoff Volumes against Rainfall Depths for Parallel Pipes and the Combined at Bothwell Street

FIGURE 10.34 REX PARK

RUNOFF VOLUME (RUNVOL)
VS RAINFALL DEPTH (P)

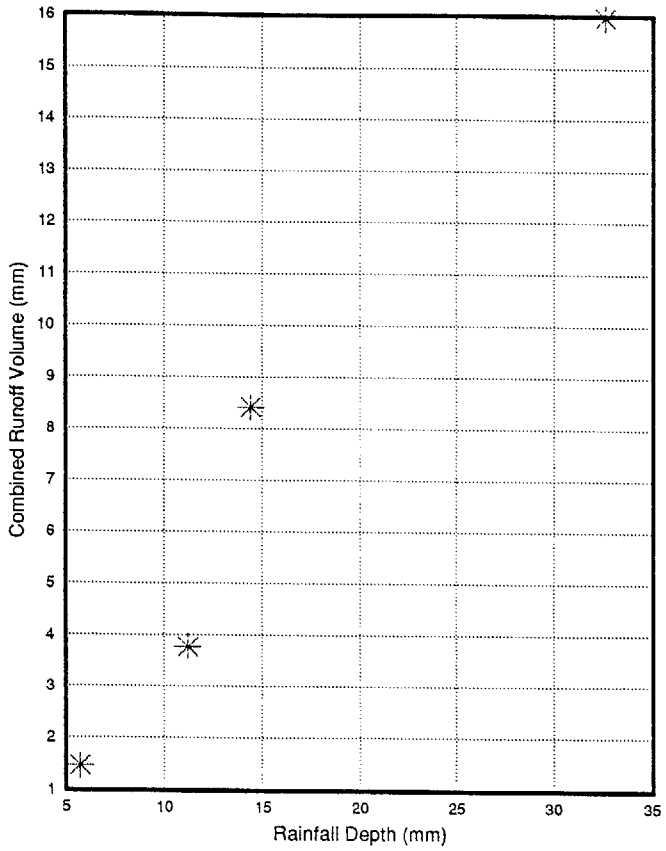


FIGURE 10.35 MILL ROAD

RUNOFF VOLUME (RUNVOL)
VS RAINFALL DEPTH (P)

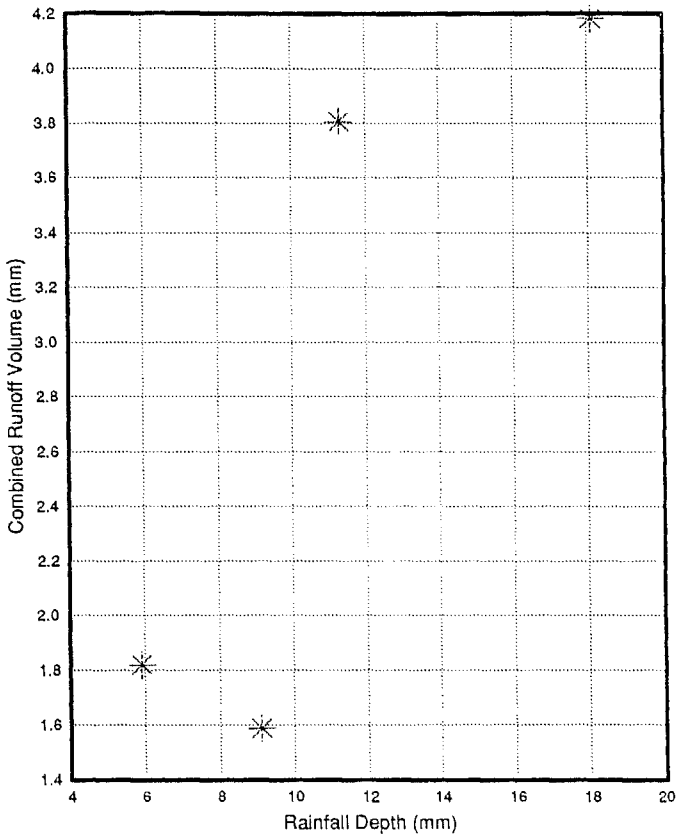


FIGURE 10.36 BOTHWELL STREET

RUNOFF VOLUME (RUNOFF)
VS RAINFALL DEPTH (P)

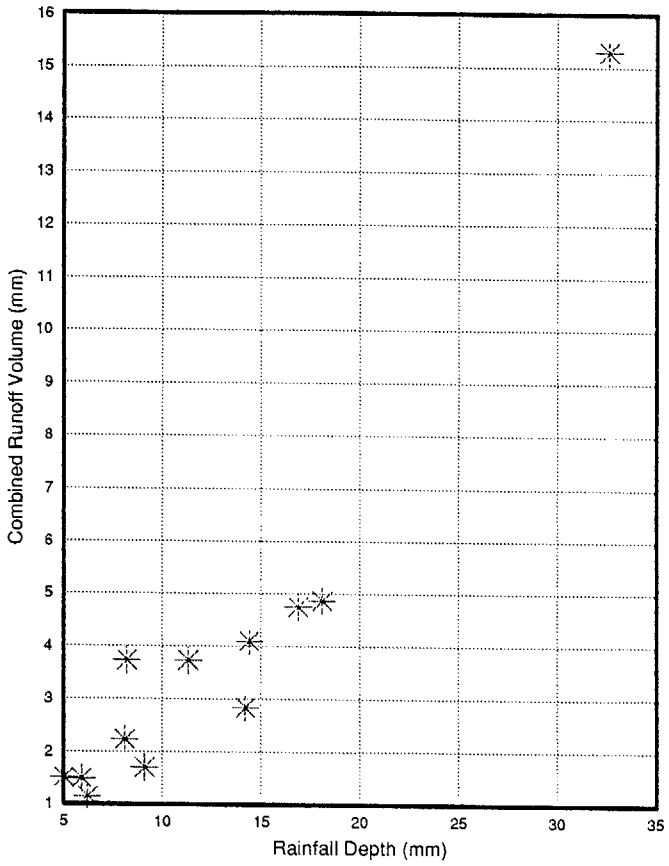
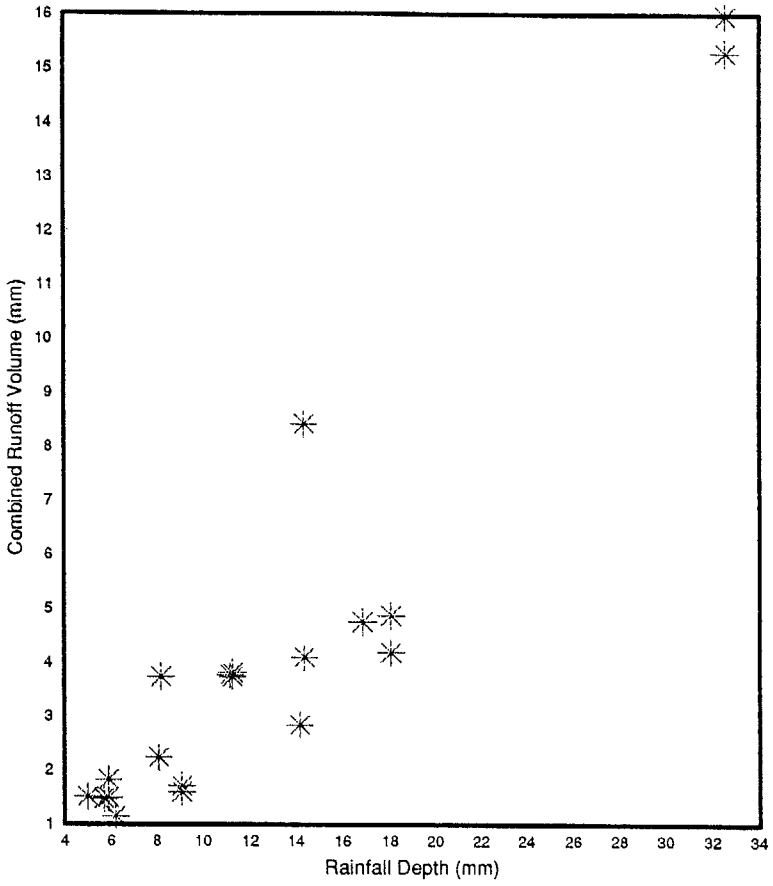


FIGURE 10.37 RUNOFF VOLUME (RUNVOL)
VS RAINFALL DEPTH (P)
FOR ALL THREE LOCATIONS



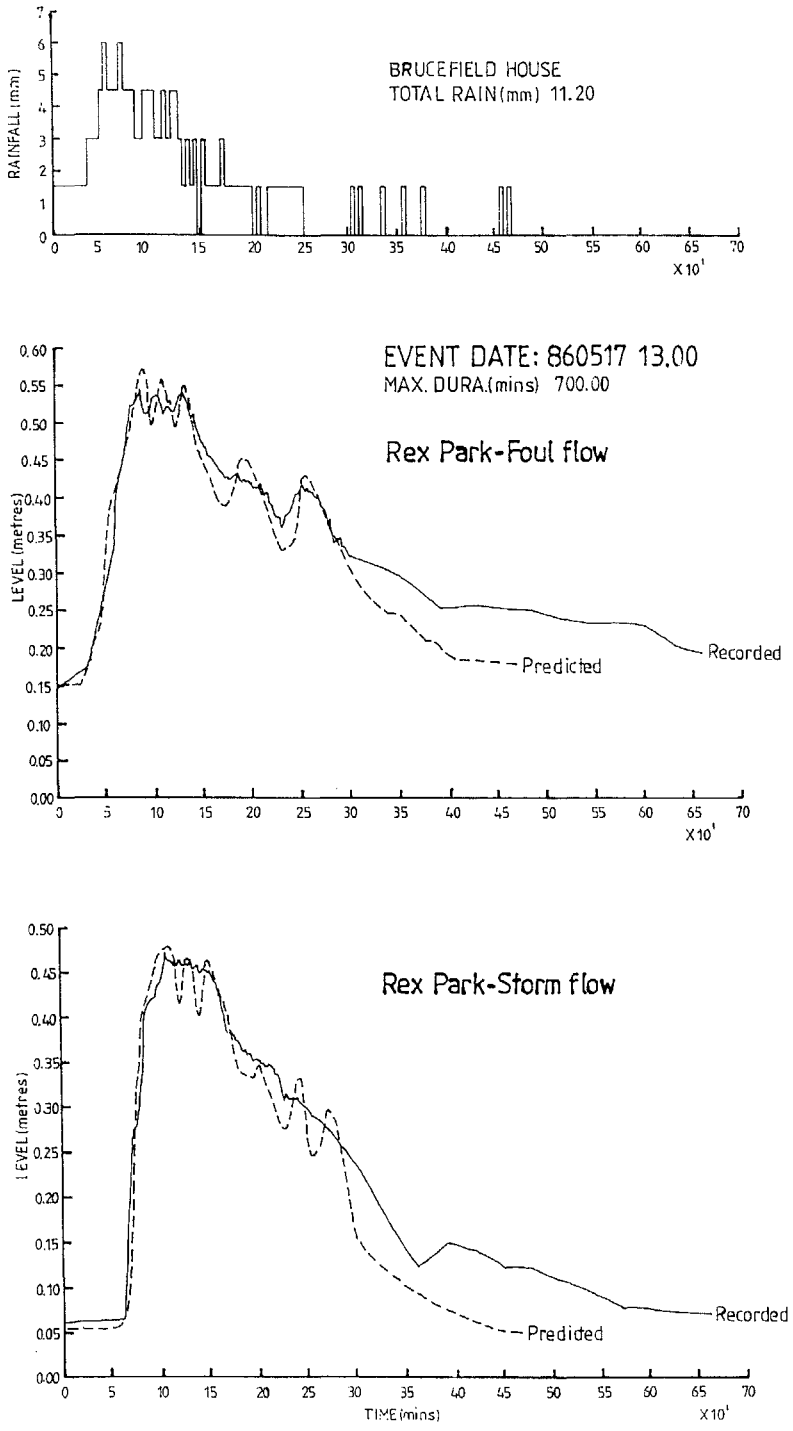


FIGURE 10.38 Comparison of Observed and Predicted Levels for Parallel Pipes at the Outfalls of Rex Park

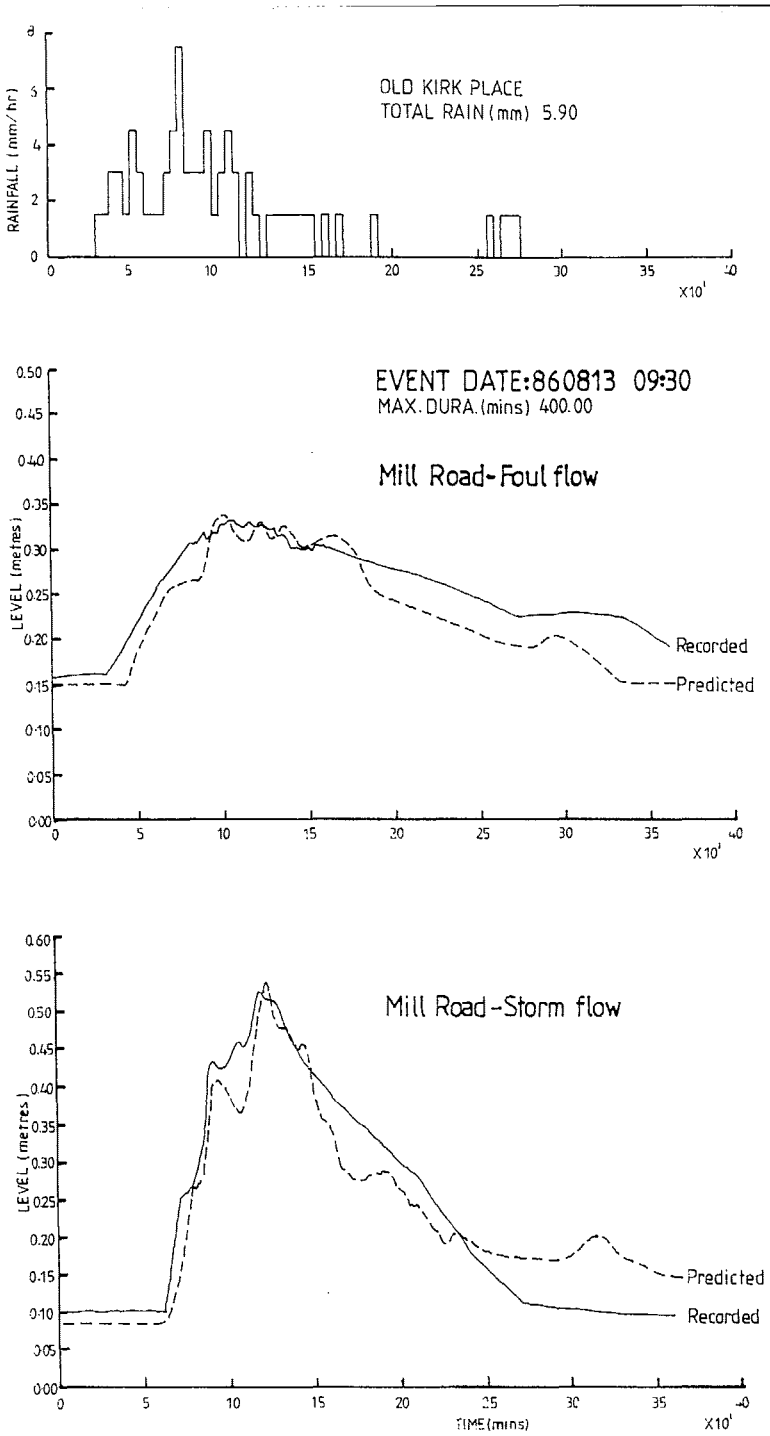


FIGURE 10.39 Comparison of Observed and Predicted Levels for Parallel Pipes at the Outfalls of Mill Road

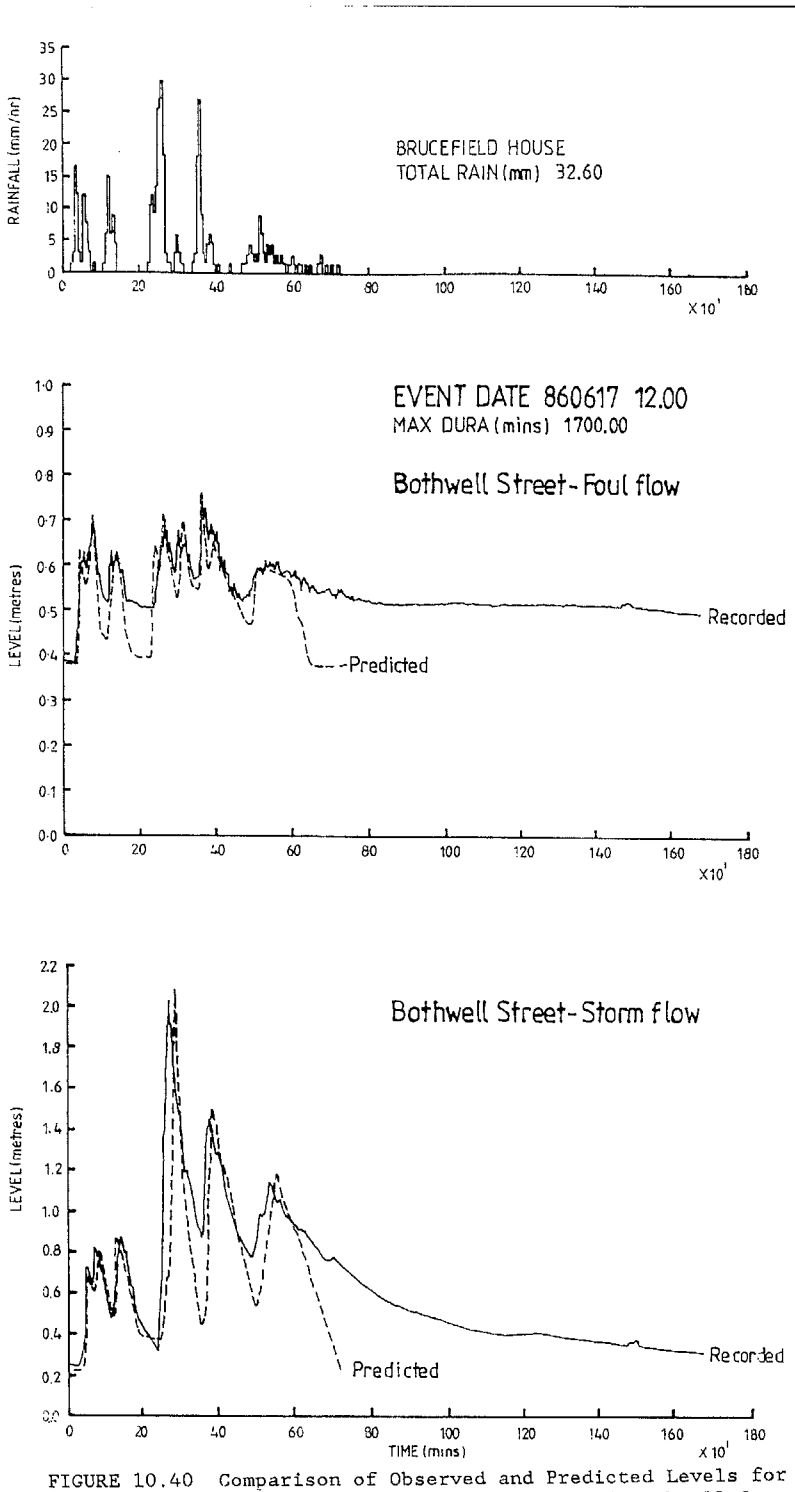


FIGURE 10.40 Comparison of Observed and Predicted Levels for Parallel Pipes at the Outfalls of Bothwell Street

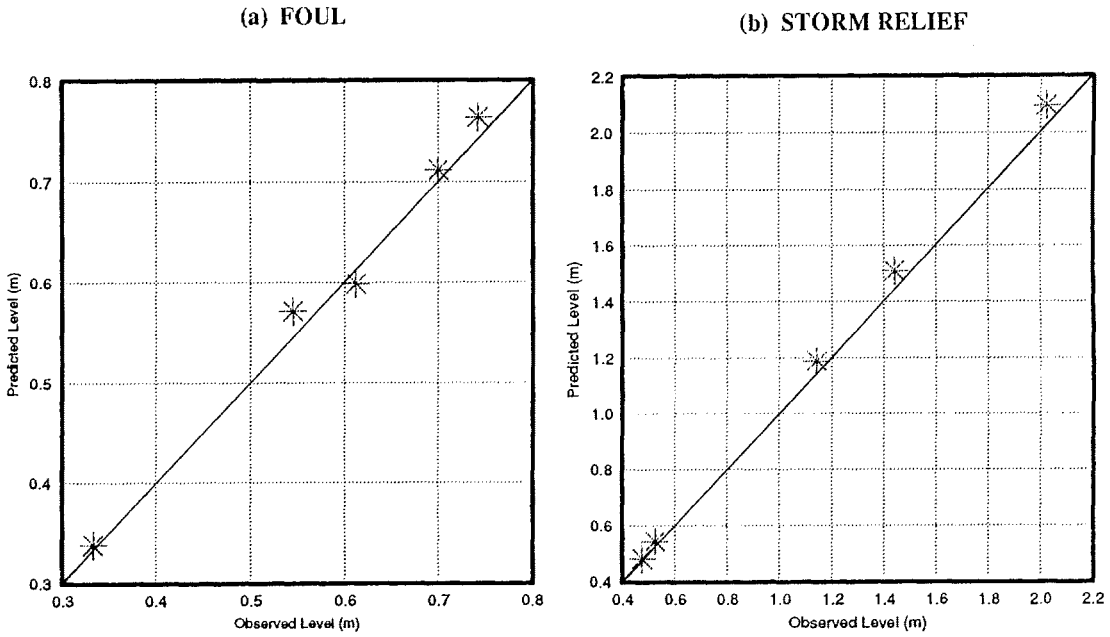


FIGURE 10.41 Predicted against Observed Peak Levels for Parallel Pipes Based on the Results of the Three Level Hydrographs

CHAPTER 11
OVERVIEW OF THE MODEL AND CONCLUSIONS

11.1 INTRODUCTION

A parallel pipe model DUPPERS has been constructed and successfully applied to the twin pipe system of Lyneburn catchment. The model predicts flow and level for each of the parallel pipes and deals with the complex flow behaviour which occurs at cross-connections.

Section 11.2 reviews the theoretical considerations of Chapters 8 and 9 in conjunction with the results of applying the model to the Lyneburn catchment described in Chapter 10. The application of the model is appraised, together with its general usage, and the points of primary importance detailed in previous chapters are emphasised.

Conclusions developed from the work and some suggestions for further work are presented in the final section.

11.2 APPRAISAL OF DUPPERS MODEL

Mathematical techniques for the numerical modelling of urban drainage runoff have been available for approximately twenty years, and have been applied successfully to a wide variety of engineering problems. In the beginning, their analysis was rather limited and often very rudimentary, and focused only on specific systems or locations. However, it is not until relatively recently that these analytical techniques have been written into widely available computer packages and extensively applied to a range of problems. Until the advent of simulation models, problems of under-capacity were solved by expanding existing sewers utilising new construction. An example of this over-provision in the past is the duplication of pipework in the Lyneburn catchment. Mathematical modelling is now accepted to be one of the most economical and effective means available to investigate problems in sewerage

systems. The fields of investigation which make use of simulation models include the examination of performance of in-sewer hydraulic structures such as storage tanks, storm overflows and pumping stations, and more commonly, the evaluation of sewer renovation and rehabilitation options (WRc/WAA 1986) by identifying the critical sewers in a system.

Most computer simulation models were developed exclusively for specific systems and are, as a result, rather limited in their application. Commercial packages, on the other hand, were developed to deal with more typical and general systems. However, they very often are far too general to be applied to those complicated networks such as Lyneburn parallel pipe system with its cross-connections. Very often simplification and modification of data is required when using the commercial packages. An example of simplification used in this study was the lumped pipe model for combining the twin pipes. A new simulation model to deal with this parallel pipe system with cross-connections was required and the result of this work is DUPPERS.

DUPPERS is a conceptual rather than empirically based model and therefore it is not constructed for a specific system. Although it is tailored to the Lyneburn system, it is generally applicable to any parallel pipe system with or without cross-connections. In the absence of such connections, DUPPERS has the potential to deal with looped or triple pipe systems.

DUPPERS employs separate sub-models to deal with the different stages in the runoff process including overland and below-ground flow routing procedures. Development of the parallel pipe model was based upon the existing DUCTS model without alteration to the above-ground runoff phase. However, the procedures for dealing with flow separation in the parallel pipes and the flow regimes in the cross-connections are completely new and employ the hydraulic equations detailed in Chapter 8.

A series of tests using various synthetic rainfall inputs were carried out to check the performance and robustness of the enhanced

components in DUPPERS. The model was then subjected to further tests using uniform rainfall to identify performance under steady state conditions before being finally verified using observed events.

The results from the synthetic rainfalls showed that the enhanced algorithms such as those for free-surface and pressurised flows in the parallel pipes, and the various flow regimes in cross-connections work satisfactorily. Stable runoff through the outfalls was obtained with the model subjected to uniform rainfall. Both flow and level hydrographs predicted by DUPPERS were found to be in close agreement for each parallel pipe when compared with the observed data. The range of percentage errors for the peak flowrates and volume of discharges were found to be within ±20% whilst a smaller range of ±5% was found for maximum levels when comparing predicted with the observed values.

The tests and comparisons confirm that the model works satisfactorily and DUPPERS is considered to be valid in its ability to model the separate flows in each of the parallel pipes. The DUPPERS model therefore simulates successfully the Lyneburn parallel pipe system.

11.3 GENERAL CONCLUSIONS

(1) A parallel pipe computer simulation model DUPPERS has been constructed, tested and applied successfully to a section of parallel pipe system in the Lyneburn drainage network.

(2) A single pipe model has also been built for the study catchment by combining the twin pipes into one. The models which use both WASSP and DUCTS give a combined outflow at the Bothwell Street overflow chamber.

(3) Appropriate hydraulic equations representing side-weir overflows including reverse flow have been applied in the model to predict flows in cross-connections and parallel pipes.

(4) The unique numbering system for the parallel pipes (500, 600) and cross-connections (700, 800) were found to be applicable and represent successfully the twin pipe systems.

(5) A procedure has been developed to determine depths of flow under free surface conditions in the system. This operates satisfactorily and predicts flow depth in pipes correctly under uniform flow condition. The computed depths for both free surface and surcharged conditions will then be printed in the output data files.

(6) System simplification has been effectively applied in reducing the number of pipes and nodes in this large system. The sewered sub-area (SSA) model has been successfully used in the simplification of the Lyneburn system.

(7) Overland catchment parameters such as contributing areas, percentages of paved, permeable and roofed areas and slopes have been verified by the lumped pipe model and effectively used in the parallel pipe model.

(8) Below ground sewer survey has been found to be essential in finding missing section of sewers and updating drainage system details.

(9) Sequential data monitoring has been found to be effective in capturing flow data at various locations for limited time durations, especially when only few flow survey loggers are available.

(10) Storm movement has little effect on the Lyneburn study catchment and rainfalls have been found to be consistent across the catchment.

(11) Urban drainage modelling cannot be entirely systematic with limited data availability. A knowledge of the study catchment as well as mathematical modelling techniques is considered a necessity for effective model construction and interpretation.

(12) Continuous monitoring of flow in sewers to capture storms, particularly those which occur rarely yet cause severe surcharge in the system, would be extremely useful for better model verification and to establish return periods of flow for the catchment concerned.

11.4 CONCLUSIONS ON SIMULATION RESULTS

The following conclusions are based on the simulation results from the DUPPERS parallel pipe model and comparisons with observed data :

(1) Synthetic rainfalls, either in uniform or varied intensities, are effective for testing the performance of computer simulation models.

(2) The model which has been tested with different synthetic input data has been shown to give reliable and stable results.

(3) Each component has been tested separately and found to be operating satisfactorily.

(4) The parallel pipe model DUPPERS operates under both free surface and surcharged flow.

(5) Surcharged sub-systems have been found to be correctly formed at different parts of the parallel pipe system.

(6) Close agreement was obtained between the observed and predicted discharge and maximum level hydrographs.

(7) The percentage differences between observed and predicted data were mostly within ±20% with a few points exceeding this range due mainly to the failure of the flow loggers to trigger during events. The percentage differences between peak observed and computed depths were within ±5%.

(8) The average values of percentage runoff (PR) for the Lyneburn catchment was found to be 61% by the model DUPPERS. This is however, higher than the 45% which has been derived using observed data. The smaller observed PR values probably results from insufficient captured data. Nevertheless, the depression storage (DEPSTOG) value of 3mm is the same for both the model prediction and the observed value.

11.5 SUGGESTIONS FOR FURTHER WORK

Both DUCTS and DUPPERS utilise the Muskingum-Cunge method of free-surface flow routing as recommended in the Flood Studies Report (NERC 1975) and HRS Report (HRS 1981). However, it is essential that both the time step and length of reach are chosen carefully to ensure that errors are within acceptable limits. It would therefore be particularly useful to examine the terms $1/\omega r$ and ξ (Conditions 3.32 and 3.33) which are both space, time and discharge dependent in the system data checking file to ensure the percentage of errors are less than 5% as recommended. It is, therefore, recommended that computing procedure should be included to readjust the length of reach (space) for those exceeding the error ranges. It would also be useful to show the constants $1/\omega r$ and ξ in the checking output data file against the sewer length. In order to achieve the above, a reach of sewer length of a system with available observed flow data should be chosen and investigated.

It is recommended that the application of DUPPERS to other catchments similar to the Lyneburn twin pipe system be carried out. This exercise will further ensure that the performance of the computer model is robust. DUPPERS is an ideal tool for the investigation of complicated flow phenomena in systems with parallel pipes and similar configurations.

At the present time, bridging pipes are treated as flow-carrying pipes by DUPPERS, flows in these pipes under both free-surface and pressurised conditions are simply carried over without routing due principally to the fact that the bridging pipes in the study

network are short in length. In simulating parallel pipe systems with long bridging pipes, a routing procedure for the bridging pipe is recommended. The additional flow routing procedure would provide a better flow representation particular for the free-surface condition.

The coefficient of discharge (Cd) for both single- and double-sided weirs may be defined by the user in preference to the default value. However, this option has not been used and tested in the Lyneburn system due to the lack of observed discharge and depth data within any of the cross-connection pipes. It is suggested that flows should be monitored at the cross-connections, especially on the foul overflow pipe, so that a better understanding of the Cd values for both types of weirs can be established. This should also prove to be extremely valuable data on the performance of low side weir overflows in general.

A final goal of future research should be the further enhancement of DUPPERS to allow simulation of other types of overflow structures such as vortex and shaft overflows. It will also be a valuable task to predict the effect on the flows in the parallel pipes by altering the overflow mechanism in the Lyneburn system. For example, closure of some or all bridging pipes, replacement of the weir overflow by other hydraulic structures. Another potential enhancement to the DUPPERS model would be the simulation of looped or even triple pipe system against recorded flow data.

REFERENCES

(1) Abbott M.B. (1966) An Introduction to the Method of Characteristics, Thames and Hudson Publishing, London.

(2) Abbott M.B. (1975) Method of Characteristics, Chapter 3 of Unsteady Flow in Open Channels, Yevjevich V. & Mahmood K. (Ed), Water Resources Publications, Colorado.

(3) Abbott M.B. & Cunge J.A. (1982) (Ed) Engineering Applications of Computational Hydraulics, Volume 1, Pitman Books Limited, London.

(4) Abbott M.B. & Ionescu F. (1967) On the Numerical Computation of Nearly-Horizontal Flows, Journal of Hydraulic Research, Volume 5, No. 2, pp 97-117.

(5) Abbott M.B. & Verhoog F.H. (1968) Etude Numerique des Ondes de Crue Dans un Fleuve Avec Plaines d'Inondation (in French), Journal of Hydraulics, Paris, Question II, Report 11.

(6) Ackers P. (1957) A Theoretical Consideration of Side Weirs as Storm-Water Overflows, Proc. of Meeting of Hydraulics Engineering Division, Instn. of Civil Engineers, Hydraulic Paper No. 11, Volume 6, Feb. 1957, pp 250-269.

(7) Ackers P. (1959) An Investigation of Head Losses at Sewer Manholes, Paper presented in Civil Eng. and Public Works Review, Volume 54, pp 882-884 & 1033-1036.

(8) Ackers P. (1969) Charts for the Hydraulic Design of Channels and Pipes, Hydraulic Research Paper No. 2, Third Edition, Hydraulics Research Station, HMSO.

(9) Ackers P. & Harrison A.J.M. (1964) The Attenuation of Flood-Waves in Part-Full Pipes, Proc. of ICE, Volume 28, pp 361.

(10) Ackers P., Harrison A.J.M. & Brewer A.J. (1967) Laboratory Studies of Storm Overflows with Unsteady Flow, Paper 4 in Symp. on Storm Sewage Overflows.

(11) Akan A.O. (1985) Similarity Solution of Overland Flow on Pervious Surface, Journal of Hydraulics Eng., Vol. 111, No. 7, ASCE, July 1985.

(12) Akan A.O. & Yen B.C. (1981) Mathematical Model of Shallow Water Flow over Porous Media, Journal of the Hydraulics Division, ASCE, Volume 107, No. HY4, April 1981, pp 479-494.

(13) Akan A.O. & Yen B.C. (1984) Effect of Time Distribution of Rainfall on Overland Runoff, Proc. of the Third Int. Conf. on Urban Storm Drainage, Chalmers University of Technology, Goteborg, Sweden, 4-8th June 1984.

(14) Ali K.H.M. & Pateman D. (1980) Theoretical and Experimental Investigation of Air-Regulated Siphons, Proc. of Inst. of Civil Engineers, Part 2, Volume 6, March 1980, London.

(15) Allen J.W. (1957) The Discharge of Water over Side Weirs in Circular Pipes, Proc. of Meeting of Hydraulics Engineering Division, Instn. of Civil Engineers, Hydraulic Paper No. 12, Volume 6, Feb. 1957, pp 270-287.

(16) Amein M. (1968) An Implicit Method for Numerical Flood Routing, Water Resources Research, Volume 4, No. 4.

(17) Amein M. & Chu H.L. (1975) Implicit Numerical Modelling of Unsteady Flows, Journal of Hydraulics Division, ASCE, Volume 101, No. HY6.

(18) Amein M. & Fang C.S. (1969) Streamflow Routing (with Applications to North Carolina Rivers), Report No. 17, North Carolina Water Resources Research Institute, Raleigh, North Carolina.

(19) Amein M. & Fang C.S. (1970) Implicit Flood Routing in Natural Channels, Journal of Hydraulics Division, ASCE, Volume 96, No. HY12, December.

(20) Amorocho J. & Wu B. (1977) Mathematical Models for the Simulation of Cyclonic Storm Sequences and Precipitation Fields, Journal of Hydrology, Volume 32, pp 329-345.

(21) Angus D.G.K (1985) Test of the DUCTS Version 1.0 Program, HND Project Report, Dept. of Civil Engineering, Surveying and Building, Dundee Institute of Technology.

(22) Archer B., Bettess F. & Colyer P.J. (1978) Head Losses and Air Entainment at Surcharged Manholes, Report No. IT 185, Nov. 1978, Hydraulics Research Station, England.

(23) Ashley R.M. (1985) Plotting of S.W.A. Flow Data & Comparison of Result, Internal papers, Dundee Institute of Technology.

(24) Ashley R.M. (1986) Comments on the Discussion Document - Guide to Sewer Flow Syrveys, Report No. ER 175E, WRc, Internal Report, July 1986, Dundee Institute of Technology.

(25) Ashley R.M. & Jefferies C. (1983) Dunfermline Sewerage System -- Initial Report, Basis of Computer Model and Corresponding Data Requirement, Dundee Institute of Technology, July 1983.

(26) Ashley R.M. & Jefferies C. (1984) DUCTS - Application of the Computer Simulation Model to Dunfermline, Report No. DCT/FRC/2/2, Dept. of Civil Eng., Surveying and Building, Dundee Institute of Technology.

(27) Ashley R.M. & Jefferies C. (1986) DUCTS - User's Manual, Internal Report, August 1986, Dundee Institute of Technology.

(28) Ashley R.M., Jefferies C. & Goodison M.J. (1989) Sewer Simulation in Tayside and Fife, Paper presented to ICE Meeting at Dundee Institute of Technology, 17th October 1989.

(29) Ashley R.M., Jefferies C. & Stevens G.S.W. (1986) Overloading of the Dunfermline Sewerage System due to Peripheral Urban Expansion, Proc. of ICE Centenary Conf. on Infrastructure Renovation and Waste Control, April 1986, UMIST.

(30) Ashworth R.G. & O'Flaherty C.A. (1974) Validity of the Modified Bilham Equation, Journal of Instn. Municipal Engineers, Volume 101, pp 43-50.

(31) Au-Yeung W.K. (1985) Detectionic Logger Calibration -- Experiments & Results, Internal paper, May 1985, Dundee Institute of Technology.

(32) Au-Yeung W.K. (1985[1]) Computer Simulation of Storm Sewer Flows in Dunfermline, Research Report - One Year On, Internal Report, Oct. 1985, Dundee Institute of Technology.

(33) Au-Yeung W.K. (1985[2]) Raingauges Calibration - Experiments and Results, Internal Paper, July 1985, Dundee Institute of Technology.

(34) Au-Yeung W.K. (1986) Computer Simulation of Storm Sewer Flows in Dunfermline - Progess Summary Report, April 1986, Dept. of Civ, Eng., Sur. and Bldg., Dundee Inst. of Technology.

(35) Au-Yeung W.K. & Ashley R.M. (1985) Discussions on the Flow Logger Calibration and test results, Verbal Discussions, Dundee Institute of Technology.

(36) Au-Yeung W.K., Jefferies C. & Ashley R.M. (1986) The Lyne Burn Case Study, Paper presented to the WaPUG Autumn Meeting, the City Chambers, Glasgow, 6th Nov. 1986.

(37) Au-Yeung W.K., Ashley R.M., Goodison M. & Jefferies C. (1989) The Development of Flows Simulation Models for Urban Catchments in North East Scotland, Proc. of Int. Conference on Computational Modelling & Experimental Methods in Hydraulics, HydroComp 13-16 June 1989, Dubrovnik, Yugoslavia.

(38) Babbitt H.E. (1953) Sewerage and Sewage Treatment, Wiley, 7th Edition, New York.

(39) Balmforth D.J. (1978) Flow over Side Weirs, PhD Thesis, University of Sheffield.

(40) Balmforth D.J. (1982) Improving the Performance of Stilling Pond Storm Sewage Overflow, Proc. of the First Int. Seminar on Urban Drainage Systems, University of Southampton, England, Sept. 1982.

(41) Balmforth D.J. (1985[1]) Storm Sewage Overflows and Sewerage Rehabilitation, Paper presented in the Public Health Engineer, Volume 13, No. 3, July 1985.

(42) Balmforth D.J. (1985[2]) Design of Storm Sewage Overflow Structures, Paper presented in One-Day Course on Research Developments in Sewerage Design and Operation at Trent Polytechnic, Nottingham, 28th June 1985.

(43) Balmforth D.J. & Henderson R.J. (1988) A Guide to the Design of Storm Overflow Structures, Report No. ER 304E, April 1988, WRc, Swindon, UK.

(44) Balmforth D.J. & Sarginson E.J. (1977) Discussion of Experimental Investigation of Flow over Side Weirs, Reported by El-Khashab and Smith, Proc. of ASCE, Journal of Hydraulics Division, No. HY8, pp 941-943.

(45) Balmforth D.J. & Sarginson E.J. (1978) A Comparison of Methods of Analysis of Side Weir Flows, Paper presented in Chartered Municipal Engineer, Oct. 1978.

(46) Balmforth D.J. & Sarginson E.J. (1983) An Experimental Investigation of the Discharge Capacity of Side Weirs, Int. Conf. on Floods and Flood Control, BHRA, London, Sept. 1983.

(47) Balmforth D.J., Lea S.J. & Sarginson E.J. (1984) Development of a Vortex Storm Sewage Overflow with Peripheral Spill, Proc. of the Third Int. Conf. on Urban Storm Drainage, Chalmers Univ. of Tech., Goteborg, Sweden, 4-8th June 1984.

(48) Balmforth D.J., Milroy C.A.J. & Sarginson E.J. (1982) Development of a Compact Air-Regulated Siphon for Storm Sewage Overflows, Proc. of the First Int. Seminar on Urban Drainage Systems, University of Southampton, Sept. 1982.

(49) Barr D.I.H. (1980) The Transition from Laminar to Turbulent Flow, Proc. of Instn. of Civil Engineers, Part 2, Volume 69, June 1980, pp 555-562.

(50) Barr D.I.H. (1981) Solutions of the Colebrook-White Function for Resistance to Uniform Turbulent Flow, Proc. of Instn. of Civil Engineers, Part 2, Volume 71, June 1981, pp 529-535.

(51) Barr D.I.H. (1986) Channel Transform Approach for Explicit Colebrook-White Solutions for Partially Full Pipes, Proc. of Instn. of Civil Engrs., Part 2, Volume 81, March 1986, pp 81-94.

(52) Bell F.C. (1969) Generalised Rainfall-Duration Frequency Relationships, Proc. of American Society of Civil Engineers, Journal of Hydraulica Division, Vol. 95, No. NY1, pp 311-327.

(53) Bell F.C. (1976) The Areal Reduction Factor in Rainfall Frequency Estimation, Report No. 35, pp 58, Institute of Hydrology, Wallingford.

(54) Beron P., Briere F., Roussell J. & Riley J.P. (1984) An Evaluation of Some Real-Time Techniques for Controlling Combined Sewer Overflows, Proc. of Third Int. Conf. on Urban Storm Drainage, Chalmers University of Technology, Goteborg, Sweden, 4-8th June 1984.

(55) Bettess R. & Price R.K. (1976) Comparison of Numerical Methods for Routing Flow along a Pipe, Report No. IT 162, Hydraulics Research Station, Wallingford, October 1976.

(56) Bettess R. & Price R.K. (1978) A Mathematical Surcharging Model for Storm Sewer Systems, Report No. IT 178, Hydraulics Research Station, Wallingford.

(57) Bettess R., Pitfield R.A. & Price R.K. (1978) A Surcharging Model for Storm Sewer System, Proc. of Int. Conf. on Urban Storm Drainage, Helliwell P.R. (Ed), Southampton, London, Pentech Press, pp 306-316.

(58) Bilham E.G. (1935) Classification of Heavy Falls in Short Period, British Rainfall 1935, pp 262-280, HMSO, London.

(59) Blyth K. & Kidd C.H.R. (1977) The Development of a Meter for the Measurement of Discharge Through a Road Gully, Paper presented in Chart. Municipal Eng., Volume 109, No. 2, pp 24-27.

(60) Boussinesq J.V. (1877) Essay on the Theory of Water Flow, Academie des Sciences, Paris, Vol. 23, Ser. 2, No. 1, pp 1-680.

(61) Braine C.D.C. (1947) Drawn-Down and other Factors to the Design of Stormwater Overflows, Journal of the Inst. of Civil Engineers, Volume 28, April 1947, pp 136-163.

(62) Braine C.D.C. (1957) Design and Operation of Low-Head Self Priming Siphons, Paper presented in The Surveyor, 2nd Nov. 1957.

(63) Brandstetter A. (1975) Assessment of Mathematical Models for Storm and Combined Sewer Management, Environmental Protection Agency, Cincinatti, Ohio.

(64) Burrows R. & Ali K.H.M. (1982[1]) The Shaft Overflow : A Storm Sewerage Overflow Device Appropriate to Differential Head Applications, Journal of Inst. of Municipal Engineers, Volume 109, June 1982, pp 143-148.

(65) Burrows R. & Ali K.H.M. (1982[2]) Stormwater Overflow : Interpretation of Particle Retention Test Results, Proc. of the First Int. Seminar on Urban Drainage System, University of Southampton, Sept. 1982.

(66) Burrows R., Ali K.H.M. & Pateman D.R. (1984) A Circular Shaft Overflow Chamber for Large Interceptor Schemes, Proc. of the Third Int. Conf. on Urban Storm Drainage, Chalmers University of Technology, Goteborg, Sweden, 4-8th June 1984.

(67) Burrows R., Crowder G. & Lowther J. (1989) Sewer Flow Surveys : An Appraisal of Combined Velocity / Depth Logger Performance by Laboratory Studies, Proc. of Conf. on Drainage and Waste Management into the 1990s, 9-11th May 1989, Stakis Earl Grey Hotel, Dundee.

(68) Butter N. (1986) A Hydrological and Hydraulic Analysis of Highway Culverts, Honours Project Report 1985-1986, Dept. of Civ. Eng., Sury. and Bldg., Dundee Inst. of Technology.

(69) Calder I.R. & Kidd C.H.R. (1978) A Note on the Dynamic Calibration of Tipping-Bucket Gauges, Journal of Hydrology, Volume 39, June 1978, pp 383-386.

(70) Catterson G. (1986) Bolton Central Area Model Verification, Paper presented to the WaPUG Autumn Meeting, the Council Chambers, Glasgow, 6th Nov. 1986.

(71) Chaplin R. (1985) Experience with Storm Overflows, Paper presented to the WaPUG Autumn Meeting, 5th Nov. 1985, Birmingham City Council, England.

(72) Chapman R. (1990) Design of Sewer Systems Incorporating Runoff Controls, Symposium on Developments in Storm Drainage : A Symposium on Infiltration and Storage of Stormwater in New Developments, 3 & 4 Jan. 1990, Polylink, Sheffield City Polyte.

(73) Chen C.W. & Shubinski R.P. (1971) Computer Simulation of Urban Storm Water Runoff, Proc. Amer. Soc. Civil Engrs., Journal of Hydraulics Division, Volume 97, No. HY2, pp 289-301.

(74) Cheverean G., Holly F. & Preissman A. (1978) Can Detailed Hydraulic Modelling be Worthwhile When Hydrologic Data is Incomplete?, Proc. of the First Int. Conf. on Urban Storm Drainage, University of Southampton, England, April 1978.

(75) Chow V.T. (1959) Open Channel Hydraulics, McGraw-Hill Book Company Inc., New York.

(76) CIRIA (1974) Rainfall, Runoff and Surface Water Drainage of Urban Catchments, Construction Industry Research and Information Association, London.

(77) Clifforde I.T. (1984) The Assessment of Sewer System Hydraulic Performance, Proc. of Int. Conf. on the Planning, Construction, Maintenance and Operation of Sewerage Systems, University of Reading, England, 12-14th Sept. 1984.

(78) Clifforde I.T. & Price R.K. (1986) Quality Modelling in Urban Storm Sewers - Current Developments, Proc. of Symp. on Developments in Storm Sewerage Management, Sheffield City Polytechnic, UK.

(79) Coleman G.S. & Smith D. (1923) The Discharging Capacity of Side Weirs, Proc. of Instn. of Civil Engineers, Selected Engineering Paper No. 6, London.

(80) Collinge V.K. (1957) The Discharge Capacity of Side Weirs, Proc. of Meeting of Hydraulics Engineering Division, Paper No. 13, Instn. of Civil Engrs., Volume 6, Feb. 1957, pp 288-304.

(81) Colyer P.J. (1977) Performance of Storm Drainage Simulation Models, Proc. Instn. Civil Engrs., Part 2, Volume 63, June 1977, pp 293-309.

(82) Colyer P.J. (1981) Variation of Rainfall Over an Urban Catchment, Proc. of the Second Int. Conf. on Urban Storm Drainage, Urban Stormwater Hydraulics and Hydrology, Yen B.C. (Ed), Urbana, Illinois, USA, 15-19th June 1981.

(83) Colyer P.J. (1982) Rainfall-Runoff Data Collection in Urban Drainage Catchments, Report No. IT 232, June 1982, Hydraulics Research Station, Wallingford.

(84) Colyer P.J. & Pethick R.W. (1976) Storm Drainage Design Methods -- A Literature Review, Report No. INT 154, 4th Edition, Sept. 1977, Hydraulics Research Station, Wallingford.

(85) Colyer P.J. & Price R.K. (1981) Experience in the Use of a Computer Program to Simulate Sewer Networks under Surcharge, Proc. on Restoration of Sewerage Systems, ICE Conf. on Urban Storm Drainage, 1981.

(86) Cordery I. (1976) Some Effects of Urbanisation on Streams, Civil Engineering Trans., Instn. Engineers Austr., CE18(1), pp 7-11.

(87) Cunge J.A. (1969) On the Subject of a Flood Propagation Computation Method (Muskingum Method), Journal of Hydraulic Research, Volume 7, No. 2, pp 205-230.

(88) Cunge J.A. (1969) On the Subject of a Flood Propagation Method, Journal of Hydraulic Research, IAHR, No. 7, pp 205-230.

(89) Cunge J.A. (1974) Evalution Problem of Storm Water Routing Mathematical Models, Water Resources Res., Volume 8, pp 1083-1087.

(90) Cunge J.A. & Mazaudou (1984) Mathematical Modelling of Complex Surcharge Systems : Difficulties in Computation and Simulation of Physical Situations, Proc. of the 3rd Int. Conf. on Urban Storm Drainage (Vol. 1), Goteborg, Sweden, 4-8 June 1984.

(91) Cunge J.A. & Wegner M. (1964) Numerical Integration of the Barre de Saint-Venant Flow Equations by an Implicit Finite Difference Scheme, La Houille Blanche, No. 1, pp 33-39.

(92) Cunge J.A., Holly F.M.,Jr & Verwey A. (1980) Practical Aspects of Computational River Hydraulics, Pitman Publishing Limited, London.

(93) De Marchi G. (1934) Saggio di Teoria del Funzionamente degli Stramazzi Laterali, L'Energia Elettrica, XI, Nov. 1934.

(94) De Saint-Venant B. (1871) Theory of Unsteady Water Flow with Application to River Floods and to Propagation of Tides in River Channels, Acad. Sci. Paris, Volume 73, pp 148-154, 237-240.

(95) Delo E.A. & Saul A.J. (1989) Chart for the Hydraulic Design of High Side-Weirs in Storm Sewage Overflows, Proc. of Inst. of Civil Engineers, Part 2, Volume 87, June 1989, pp 175-193.

(96) Detectronic Ltd. (1983) Flow Surveys Unit - User's Manual, Detectronic Limited, Systems House, Edge Nook Road, Shadsworth Ind. Park, Blackburn, Lancs.

(97) DHI (1980) S11S New Development (in Danish), Danish Hydraulic Institute, August 1980.

(98) DHI (1987) MOUSE -- Modelling of Urban Sewer Systems on Micro-Computers, Danish Hydraulic Institute, Agern Alle 3, DK-2970 Horsholm, Denmark.

(99) Digital Equipment Corporation (1984) Programming in VAX Fortran, VAX/VMS V4 System, Ref. No. AA-D034D-TE, Sept. 1984, DEC, Maynard, Massachusetts, USA.

(100) Dooge J.C.I. (1973) Linear Theory of Hydrologic System, US Dept. of Agriculture, Agric. Research Service, Technical Bulletin 1468, pp 327.

(101) Eadon A.R. (1984) Effective Simulation of Existing Sewerage Systems, Proc. of Int. Conf. on the Planning, Construction, Maintenance and Operation of Sewerage Systems, University of Reading, UK, 12-14th Sept. 1984.

(102) Eadon A.R. (1986) Hydraulic Design for Sewer Rehabilitation, Paper presented to the WaPUG Autumn Meeting, the City Council Chambers, Glasgow, 6th Nov. 1986.

(103) El-Khashab A. & Smith K.V.H. (1976) Experimental Investigation of Flow over Side-Weirs, Paper presented in Journal of Hydraulic Division, ASCE, Volume 102, No. HY9, Sept. 1976, pp 1255-1268.

(104) Engels H. (1917) Mitteilungen Ausdem Dresdener Flussbaulaboratorium, ForschArb. IngWes.

(105) Engelund F.A. & Pedersen F.B. (1978) Hydraulics (in Danish), Den Private Ingeniorfond, Danmarks Tekniske, Hojskole.

(106) Ervine D.A. (1976) The Design and Modelling of Air-Regulated Siphon Spillways, Proc. of the Inst. of Civil Engineers, Part 2, Volume 61, June 1976, London.

(107) Falk J. & Niemczynowicz J. (1978) Characteristics of the Above-Ground Runoff in Sewered Catchments, Proc. of the First Int. Conf. on Urban Storm Drainage, University of Southampton, England, April 1978.

(108) Falk J. & Niemczynowicz J. (1979) Modelling of Runoff from Impermeable Surfaces, Report No. 3024, University of Lund, Dept. of Water Resources Engineering, Swenden.

(109) Felgate D.G. & Read D.G. (1975) Correlation Analysis of the Cellular Structure of Storms Observed by Raingauge, Journal of Hydrology, Volume 24, pp 191-200.

(110) Fiddes D. (1984) A Sewerage Rehabilitation Strategy for the United Kingdom, Paper presented to the Int. Conf. on the Planning, Construction, Maintenance and Operation of Sewerage Systems, University of Reading, UK, 12-14th Sept. 1984.

(111) Fiddes D. (1984) The Hydraulic Upgrading of Urban Sewer Systems, Proc. of the Third Int. Conf. on Urban Storm Drainage, Chalmers University of Technology, Goteborg, Sweden, 4-8th June 1984.

(112) Field R. (1972) The Swirl Concentration as a Combined Sewer Overflow Regulator Facility, U.S. Environmental Protection Agency, Report No. EPA-R2-72-008, Washington DC 20460, September 1972.

(113) Field R. (1974) Design of a Combined Sewer Overflow Regulator / Concentration, Journal of the Water Pollution Control Federation, Volume 46, No. 7, July 1974, pp 1722-1740.

(114) Folland C.K. & Shaw T.L. (1979) Storm Movement, Informal Discussion reported by Kelwey P.S., Proc. of ICE, Volume 66, Part 1, August 1979.

(115) Folland C.K., Kelway P.S. & Warrilow D.A. (1981) The Application of Meteorological Information to Flood Design, The Flood Studies Report -- 5 Years On, ICE, London.

(116) Fox J.A. (1977) Hydraulic Analysis of Unsteady Flow in Pipe Networks, MacMillan Press Ltd., London.

(117) Frazer W. (1957) The Behaviours of Side Weirs in Prismatic Rectangular Channels, Proc. of Meeting of Hydraulics Engineering Division, Instn. of Civil Engineers, Hydraulic Paper No. 14, Volume 6, Feb. 1957, pp 305-328.

(118) Fread D.L. (1971) Discussion of Implicit Flood Routing in Natural Channels by Amein M. & Fang C.S., Journal of the Hydraulics Division, ASCE, Volume 97, No. HY7.

(119) Frederick M.R. & Markland E. (1967) The performance of Stilling Ponds in Handling Solids, Proc. of Symp. on Storm Sewage Overflows, ICE, London.

(120) Grant (1984) Squirrel Pulse Event Recorder - User's Manual, Grant Instrument Ltd., Cambridge.

(121) Green M.J. (1985) Future Developments in Instrumentation, Paper presented to the WaPUG Autumn Meeting, 5th Nov. 1985, City Council, Birmingham.

(122) Green M.J. & Drinkwater A. (1985) Guide to Sewer Flow Surveys, Discussion Document, Report No. ER 175E, WRc, July 1985.

(123) Gunst R.J.M. & Kidd C.H.R. (1980) The Application of a Kinematic Wave Model to Urban Surfaces, Report No. 67, May 1980, Inst. of Hydrology, Wallingford.

(124) Hall M.J. (1973) The Hydrological Consequences of Urbanisation -- An Introductory Note, Proc. Res. Colloquium on Rainfall, Runoff and Surface Water Drainage of Urban Catchments, Paper 10, Construction Industry Research and Information Association (CIRIA), Bristol.

(125) Hall M.J. (1984) Urban Hydrology, Elsevier Applied Science Publishers Limited.

(126) Halliwell A.R. & Saul A.J. (1980) The Use of Hydraulic Models to Examine the Performance of Storm Sewage Overflows, Proc. of the Inst. of Civil Engineers, Part 2, Volume 69, pp 245-259, June 1980.

(127) Harding N.R. (1986) The Boreham Wood Case Study - Modelling Complex Junctions, Paper presented to the WaPUG Autumn Meeting, 6th Nov. 1986, the City Council Chambers, Glasgow.

(128) Hayami S. (1951) On the Propagation of Flood Waves, Bulletin No. 1, Disaster Prevention Research Institute, Kyoto University, Japan.

(129) Heeps D.P. & Mein R.G. (1973) An Independent Evaluation of Three Urban Stormwater Models, Monash. University, Civil Engineering, Research Report No. 4/1973, pp 92.

(130) Heeps D.P. & Mein R.G. (1974) Independent Comparison of Three Urban Runoff Methods, Journal of the Hydraulics Division, Proc. of ASCE, Volume 100, No. NY7.

(131) Helliwell P.R., Kidd C.H.R. & Lowing M.J. (1976) Estimation of the Above-Ground Runoff Hydrograph for Storm Sewer Design Purposes, Proc. of National Symp. on Urban Hydrology, Hydraulics and Sediment Control, University of Kentucky, Lexington, Kentucky, USA.

(132) Henderson F.M. (1966) Open Channel Flow, MacMillan and Co., New York.

(133) Henderson R.J. (1987) Discussions on the Coefficients of Discharge for Single and Double Side-Weirs, Verbal Discussions Reported by Jefferies C., Dundee Institute of Technology.

(134) Henderson R.J. & Moys G.D. (1987) Developments of a Sewer Flow Quality Model for the U.K., Proc. of the Fourth Int. Conf. on Urban Storm Drainage, Lausanne.

(135) HMSO (1963) A Guide for Engineers to the Design of Storm Sewer Systems, Road Note Number 35, Road Research Laboratory, DoE, London.

(136) HMSO (1970) Technical Committee on Storm Overflows and Disposal of Storm Sewage, Final Report.

(137) HMSO (1976) A Guide for Engineers to the Design of Storm Sewer Systems, Road Note Number 35, 2nd Edition, Transport & Road Research Loboratory, DoE, London.

(138) Hoff-Clausen N.E. & Asger K. (1981) cSystem 11 Sewer - A Storm Sewer Model, Proc. of the Second International Conference on Urban Storm Drainage, 15-19th June 1981, University of Illinois at Urbana-Champaign, Illinois, USA, Volume 1, pp 137-145.

(139) Holland D.J. (1964) Rain Intensity Frequency Relationships in Britain, Meteorological Office, Bracknell, Hydrological Memorandum 33, pp 28.

(140) Holland D.J. (1967) The Cardington Rainfall Experiment, Meteorological Magazine, Volume 96, pp 193-203, London.

(141) Howarth D.A. & Saul A.J. (1984) Energy Loss Coefficients at Manholes, Proc. of the Third Int. Conf. on Urban Storm Drainage, Chalmers University of Technology, Goteborg, Sweden, 4-8th June 1984.

(142) HR & D (1987) Stormwater Balancing and the Hydro-Brake Flow Control, Presentation to Tayside Regional Council, 2nd Dec. 1987, by Hydro Research and Development, Clevedon, Avon.

(143) HRS (1979) Tests on Slimed Sewers, Summary Report, Report No. 57, Hydraulics Research Station, Wallingford.

(144) HRS (1981) Design and Analysis of Urban Storm Drainage, The Wallingford Procedure, Vol.1 Principles, Methods and Practice, Standing Technical Committee Report No. 28, Sept. 1981, DoE/NWC.

(145) HRS (1983[1]) Charts for the Hydraulic Design of Channels and Pipes, 5th Edition, Hydraulics Research Station, Wallingford.

(146) HRS (1983[2]) Tables for the Hydraulic Design of Pipes, 4th Edition, Hydraulics Research Station, Wallingford.

(147) HRS (1985) Micro-WASSP Program User's Guide - The Wallingford Procedure, Volume 2, STC Report No. 29, 2nd Edition, Feb. 1985, Hydraulics Research Limited, Wallingford.

(148) HRS (1987) WALLRUS User's Manual, Wallingford Procedure Software, 3rd Edition Aug. 1989, Hydraulics Research Station.

(149) IOH (1974-1979) Studies in Urban Hydrology -- A Summary Reports of Urban Hydrology Research, December 1980, Institute of Hydrology.

(150) IOH (1981) Simple Instrument System - Raingauge, Operating Instructions, Ref. No. IGS/VW, Jan. 1981.

(151) Isaacson E., Stoker J.J. & Troesch B.A. (1954) Numerical Solution of Flood Prediction and River Regulation Problems (Ohio-Mississippi Floods), 3 Reports, Inst. of Math. Sci. Rept., No. 200 (1953), No. 205(1954), No. 235(1956), New York University, New York.

(152) Izzard C.F. (1944) The Surface Profiles of Overland Flow, VI Trans. Amer. Geophys. Union, pp 959.

(153) Izzard C.F. (1946) Hydraulics of Runoff from Developed Surfaces, Proc. of Highway Research Board, 26th Annual Meeting.

(154) Jacobsen P. (1983) Surcharged Sewer Simulation, DIF's Spildevandskomite, Copenhagen, Denmark, April 1983.

(155) Jefferies C. (1986) Sewer Flow Surveying - Procedure for Testing the Accuracy of Detectronic Loggers, Internal Report, Oct. 1986, Dundee Inst. of Technology.

(156) Jefferies C. & Ashley R.M. (1984) Application of the Wallingford Procedure Using Source Documentation, Proc. of the Seminar on the Wallingford Procedure, June 1984, Dundee Institute of Technology.

(157) Jefferies C. & Ashley R.M. (1985) The Use of Micro-Computers for the Simulation of Flows in the Dunfermline Sewerage System, Proc. of the Second Int. Conf. on Civil and Structural Eng. Computing, CIVIL-COMP 85, 3-5th Dec. 1985, ICE, London.

(158) Jefferies C. & Stevens G.S.W. (1989) Sewerage Rehabilitation for Pollution Abatment in Fife - A Review and Case Study, Proc. of Conf. on Drainage and Waste Management into the 1990s, Stakis Earl Grey Hotel, Dundee, 9-11th May 1989.

(159) Jefferies C., Ashley R.M. & Au-Yeung W.K. (1986) Final Report on WASSP Model - Joint Research Project, Report No. DCT/FRC/3/2, Dec. 1986, Dept. of Civ. Eng., Surveying and Building, Dundee Institute of Technology.

(160) Jefferies C., Au-Yeung W.K. & Goodison M.J. (1986) Internal Discussions on the Requirements and Procedures for DET Loggers Calibration, October 1986, Dundee Inst. of Technology.

(161) Jefferies C., Stevens G.S.W. & Ashley R.M. (1985) The Lyne Burn Catchment, Paper presented to the Joint Meeting of IWPC, IPHE and IWES, 20th Nov. 1985, Dundee Institute of Technology.

(162) Jefferies C., Wakelin A.W. & Ashley R.M. (1987) Keeping Track of Hydrological Data, Proc. of the 2nd Int. Conf. on Civil and Structural Engineering Computing, Civil-Comp Press 1987.

(163) Jensen M. (1981) Models for Computation of Sewer Pipe Flow, August 1981, pp 237-249.

(164) Jones D.E. Jr. (1970) Discussion of Urban Runoff by Road Research Laboratory Method, Reported by Terstriep M.L. & Stall J.B., Journal of the Hydraulics Division, ASCE, Volume 96, No. HY9, Proc. Paper 7497, Sept. 1970, pp 1879-1880.

(165) Jones S.B. (1981) Choice of Space and Time Steps in the Muskingum-Cunge Flood Routeing Method, Proc. Instn. Civ. Engrs., Part 2, Volume 71, Sept., Paper No. 8457, pp 759-772.

(166) Kidd C.H.R. (1976) A Non-Linear Urban Runoff Model, Report No. 31, April 1976, Institute of Hydrology, Wallingford.

(167) Kidd C.H.R. (1978[1]) A Calibrated Model for the Simulation of the Inlet Hydrograph for Fully Sewered Catchments, Proc. of First International Conference on Urban Storm Drainage, Southampton, pp 172-186, Pentech Press, London.

(168) Kidd C.H.R. (1978[2]) Rainfall-Runoff Processes over Urban Surfaces, Proc. of International Workshop, Report No. 53, April 1978, Institute of Hydrology, Wallingford.

(169) Kidd C.H.R & Lowing M.J. (1979) The Wallingford Urban Sub-Catchment Model, Report No. 60, Institute of Hydrology, Wallingford.

(170) Kidd C.H.R. & Helliwell P.R. (1977) Simulation of the Inlet Hydrograph for Urban Catchments, Journal of Hydrology, Volume 35, pp 159-172, Elsevier Scientific Publishing Company, Amsterdam.

(171) Kidd C.H.R. & Packman J.C. (1980) Selection of Design Storm and Antecedent Condition for Urban Drainage Design, Report No. 61, March 1980, Institute of Hydrology.

(172) Lager L.A., Shubinski R.P. & Russell L.W. (1971) Development of a Simulation Model for Stormwater Management, Journal of Water Pollution Fed., Volume 43, pp 2424-2435.

(173) Lambrecht (1982) Automatic Precipitation Gauge with Tipping-Bucket - Instructions for Use, No. 1518 H3, October 1982.

(174) Leopold L.B. (1968) Hydrology for Urban Land Planning -- A Guidebook on the Hydrologic Effects of Urban Land Use, US Geol. Survey, Circ. 554, pp 18.

(175) Liggett J.A. & Cunge J.A. (1975) Numerical Methods of Solution of the Unsteady Flow Equations, Chapter 4 of Unsteady Flow in Open Channels, Yevjevich V. & Mahmood K. (Ed), Water Resources Publications, Colorado.

(176) Liggett J.A. & Woolhiser D.A. (1967) Difference Solutions of the Shallow-Water Equation, Journal of Engineering Mechanics Division, ASCE, Volume 93, No. EM2.

(177) Lighthill M.J. & Whitham G.B. (1955) On Kinematic Waves; I -- Flood Movement in Long Rivers, Proc. Royal Society of London, Series A, 229.

(178) Linsley R.K. (1970) Discussion of Urban Runoff by Road Research Laboratory Method, Reported by Terstriep M.L. & Stall J.B., Journal of the Hydraulics Division, ASCE, Volume 96, No. HY4, Proc. Paper 7189, April 1970, pp 1100-1102.

(179) Linsley R.K., Kohler M.A. & Paulhus J.L.H. (1958) Hydrology for Engineers, McGraw-Hill Book Company Inc., New York.

(180) Luck B.N. (1985) Notes on Golden River and Detectronic Loggers, Southern Water Authority, Sussex Division, Southern Water, Brighton.

(181) Makin I.W. & Kidd C.H.R. (1979) Urban Hydrology Project : Collection and Archive of UK Hydrological Data, Report No. 59, October 1979, Inst. of Hydrology, Wallingford.

(182) Maksimovic C. & Radojkovic M. (1986) Urban Drainage Catchments, Selected Worldwide Rainfall-Runoff Data from Experimental Catchments, University of Belgrade, Yugoslavia.

(183) Marshall R.J. (1980) The Estimation and Distribution of Storm Movement and Storm Structure Using a Correlation Analysis Technique and Raingauge Data, Journal of Hydrology, Volume 48, pp 19-39.

(184) Martin C., King D., Quick N.J. & Nott N.A. (1982) Infiltration: Investigation, Analysis and the Cost / Benefit of Remedial Action, Paper 17 in Sewer Records and Monitoring Flows, Restoration of Sewerage Systems, ICE, London.

(185) Massau J. (1889) Graphical Integration of Partial Differential Equations, Assoc. Ingenieurs Sortis des Ecoles Speciales de Gand, Annales, Volume 12.

(186) Mays L.W. & Yen B.C. (1975) Optimal Cost Design of Branched Sewer System, Water Resource Res., Volume 11, pp 37-47.

(187) McCarthy G.T. (1938) The Unit Hydrograph and Flood Routing, Manuscript presented at a Conference of the North Atlantic Division, U.S. Army Corps. of Engineers.

(188) Miller W.A. & Cunge J.A. (1975) Simplifed Equations of Unsteady Flow, Chapter 5 of Unsteady Flow in Open Channels, Yevjevich V. & Mahmood K. (Ed), Water Resources Publications, Colorado.

(189) Moss G.F. (1985) Sewerage Rehabilitation : The Way Forward, the Public Health Engineer, Volume 13, No. 3, July 1985.

(190) Muzik I. (1974) Laboratory Experiments with Surface Runoff, Journal of the Hydraulics Division, Am. Soc. Civ. Engr., Volume 100, pp 501.

(191) NERC (1975) Flood Studies Report, Volume 1 to 5, Meteorological Office, National Enviroment Research Council, London.

(192) Nimmo W.H.R. (1928) Side Spillways for Regulating Diversion Canals, Trans. Amer. Soc. Civil Engineers, Volume 92, pp 1561.

(193) Norris W.H. (1948) Sewer Design and the Frequency of Heavy Rain, Proc. of Instn. Municipal Co. Engineers 75, pp 349-364.

(194) Nussey B.B. (1986) Simplification of Sub-Catchment Sewer Networks, Paper presented at the WaPUG Spring Meetings at Manchester Town Hall on 6th May 1986.

(195) NWC/DoE (1980) Sewer and Water Main Records, Standing Technical Committee on Sewers and Water Mains, Report No. 25, National Water Council and Dept. of Environment.

(196) Ong H.S. (1972) Laboratory and Numerical Studies of Runoff from a Conceptual Watershed, MSc thesis, Dept. of Civil Engineering, University of Strathclyde, Glasgow.

(197) Orman N.R. (1985) Problems with Percentage Runoff Factors in Areas with Significant Pervious Contributions, Proc. of WaPUG Meeting, Birmingham City Council, Nov. 1985.

(198) Osborne M.P. (1985) SPIDA - A New Storm Sewer Simulation Program, Hydraulics Research Limited, Wallingford, England.

(199) Packman J.C. (1986) Discussions on the Paper 'Simplification of Sub-Catchment Sewer Networks' by Nussey B.B., WaPUG Spring Meeting, Ashton Court Mansions, Bristol, 24th April 1986.

(200) Packman J.C., Cole G.A. & Robinson M. (1981) Stormwater Simulation using the Wallingford Sewered Subcatchment Model, Proc. of Second Int. Conf. on Urban Storm Drainage, Yen B.C. (Ed), 15-19th June 1981, Urbana, Illinois, USA.

(201) Papadakis C.N. & Preul H.C. (1972) University of Cincinnati Urban Runoff Model, Journal of the Hydraulics Division, ASCE, Volume 98, No. HY10, Proc. Paper 9298, pp 1789-1804.

(202) Papadakis C.N. & Preul H.C. (1973) Testing of Methods for determination of Urban Runoff, Journal of the Hydraulics Division, ASCE, Volume 99, No. HY9, Proc. Paper 9987, Sept. 1973, pp 1319-1335.

(203) Parkar M. (1983) Raingauge Calibration, Internal Report, Trent Polytechnic, Nottingham, April 1983.

(204) Parry H.J. (1983) Coping with Fife's Mining Industrial Heritage, Municipal Engineer, Volume 110, No. 7/8, July and August 1983.

(205) Pearson L.G., Thornton R.C., Saul A.J. & Howard K. (1986) An Introductory Analysis of Factors Affecting the Concentration of Pollutants in the First Foul Flush of a Combined Storm Sewer System, Proc. of Conf. on Urban Storm Water Quality and Effects upon Receiving Waters, EWPCA, Wageningen, Holland.

(206) Perkins J.A. & Gardiner I.M. (1982) Measurements of the Hydraulic Roughness of Slimmed Sewer Pipes, Report No. IT 237, July 1982, Hydraulics Research Station, Wallingford.

(207) Perkins J.A. & Gardiner I.M. (1985) The Hydraulic Roughness of Slimed Sewers, Proc. of Instn. Civ. Engrs., Part 2, Volume 79, March 1985, pp 87-104.

(208) Pratt C.J. (1986) Disposal of Storm Water by the Use of Ground Infiltration Techniques, Invited Lecture presented to Honours Students at Dundee Institute of Technology, Scotland, 14th Jan. 1986.

(209) Pratt C.J. (1987) Flow Reduction and Attenuation of Roof and Paved Surface Discharges, Use of Soakways and Porous Pavements, Paper presented at the Seminar on Flow Control in Urban Drainage Systems, 26th Nov. 1987, Hydraulics Research Station, Wallingford, England.

(210) Pratt C.J. & Balmforth D.J. (1986) Reduction and Attenuation of Surface Runoff, Paper presented at the Symp. on Developments in Storm Sewerage Management, 24-25th March 1986, Sheffield City Polytechnic, England.

(211) Pratt C.J. & Harrison J.J. (1982) Storm Runoff Simulation on a Calibrated Catchment, Proc. of the First Int. Seminar on Urban Drainage Systems, University of Southampton, England, Sept. 1982.

(212) Pratt C.J. & Henderson R.J. (1981) Overland Flow and Catchment Characteristics, Proc. of Second International Conference on Urban Storm Drainage, Urban Stormwater Hydraulics and Hydrology, June 1981, Urbana, Illinois, USA.

(213) Preissmann A. & Cunge J.A. (1961) Calcul des Intumescences sur Machines Electroniques, Proc. of 9th Congress of IAHR, Dubrovnik, pp 656-664.

(214) Price G.A., Packman J.C. & Kidd C.H.R. (1980) A Simplified Model for Sewered Catchments, Report No. 62, Institute of Hydrology, Wallingford.

(215) Price R.K. (1973[1]) Variable Parameter Diffusion Method for Flood Routing, Report No. INT 115, Hydraulics Research Station, Wallingford.

(216) Price R.K. (1973[2]) Flood Routing Methods for British Rivers, Report No. INT 111, Hydraulics Research Station, Wallingford.

(217) Price R.K. (1978) A River Catchment Flood Model, Proc. Inst. Civil Engr., Part 2, Volume 65.

(218) Price R.K. (1978[2]) Design of Storm Sewers for Minimum Construction Cost, Proc. of Int. Conf. on Urban Storm Drainage, Helliwell P.R. (Ed), Southampton, London, Pentech Press, pp 636-647.

(219) Price R.K. (1980) FLOUT -- A River Catchment Flood Method, Hydraulics Research Station, Report No. IT 168.

(220) Price R.K. (1981) A Simulation Model for Storm Sewers, Proc. of the Second International Conference on Urban Storm Drainage, Urban Stormwater Hydraulics and Hydrology, June 1981, Urbana, Illinois, USA, pp 184-192.

(221) Price R.K. & Kidd C.H.R. (1978) A Design and Simulation Method for Storm Sewers, Proc. of Int. Conf. on Urban Storm Drainage, Helliwell P.R. (Ed.), Southampton, London, Pentech Press.

(222) Price R.K. & Osborne M.P. (1986) Verification of Sewer Simulation Models, Proc. of Int. Symp. on Comparison of Urban Drainage Models with Real Catchment Data, Urban Drainage Modelling, Maksimovic C. & Radojkovic M. (ed), UDM '86 Dubrovnik, Yugoslavia.

(223) Read G.F. (1982) Sewer dereliction and Renovation - An Industrial City's View, Paper in Sewer Records and Monitoring Flows, Restoration of Sewerage Systems, ICE, London, pp 267-281.

(224) Read G.F. (1984) Sewer Renovation and Reconstruction in Manchester, Paper in the Municipal Engineer, Volume 1, June 1984, pp 27-38.

(225) Richtmyer R.D. & Morton K.W. (1967) Difference Methods for Initial Value Problems, Second Edition, Interscience Publications, New York.

(226) Robson D. (1986) A Contractor's Viewpoint on Sewerage Rehabilitation, the Public Health Engineer, Volume 14, No. 1, January 1986.

(227) Rodda J.C. (1966) A Study of Magnitude, Frequency and Distribution of Intense Rainfall in the United Kingdom, British Rainfall 1966, pp 204-215, HMSO, London.

(228) Rofe B.H. (1986) Furture Changes in UK Practice, Paper in the Symp. on Developments in Storm Sewerage Management, 24-25th March 1986, Sheffield City Polytechnic, England.

(229) Saiginson E.J. (1973) Relationship of Rainfall & Runoff in Urban Areas, Paper Presented at Rainfall, Runoff & Surface Water Drainage of Urban Catchments, Proc. of the Research Colloquium held at University of Bristol, April 1973, CIRIA.

(230) Sargent D.M. (1981) An Investigation into the Effects of Storm Movement on the Design of Urban Drainage Systems, Part 1, Public Health Engr., Volume 9, pp 201-207.

(231) Sargent D.M. (1982) An Investigation into the Effects of Storm Movement on the Design of Urban Drainage Systems, Part 2: Probability Analysis, Public Health Engr., Volume 110, pp 111-117.

(232) Saul A.J. (1977) The Performance of Storm-Sewage Chambers, PhD Thesis, Dept. of Civil Engineering, Heriot-Watt University.

(233) Saul A.J. (1985) Design of a High Side-Weir Storage Storm Sewage Overflow Related to Some Pollutant Loads in Combined Sewage, Paper presented in the One-Day Course on Research Developments in Sewerage Design and Operation, Trent Polytechnic, Nottingham, 28th June 1985.

(234) Saul A.J. (1988) Current Storm Overflow Design Practices, Types and Selection of Overflows, Paper presented at Seminar on Developments in Sewage Overflow Design and Pollution Impact Assessment, 5th May 1988, Dundee Institute of Technology.

(235) Saul A.J. (1988[1]) The Use of Storage to Attenuate Flows and Reduce Pollutant Spill, Paper presented to the Seminar on Developments in Sewage Overflow Design and Pollution Impact Assessment, 5th May 1988, Dundee Institute of Technology.

(236) Saul A.J. (1989) Combined Sewer Overflows and Storage Tank, Paper presented to the Meeting of Hydraulic Structures in Sewers, Scottish Hydraulics Study Group, 9th March 1989, University of Strathclyde, Glasgow.

(237) Saul A.J. & Delo E.A. (1981) Performance of a High-Side Weir Storm-Sewage Chamber Incorporating Storage, Proc. of the Second Int. Conf. on Urban Storm Drainage, Urbana, Illinois, USA, 15-19th June 1981.

(238) Saul A.J. & Delo E.A. (1982) Laboratory Tests on a Storm Sewer Overflow Chamber with Unsteady Flow, Proc. of the First Int. Seminar on Urban Drainage Systems, University of Southampton, England, Sept. 1982.

(239) Saul A.J. & Howarth D.A. (1982) Experimental Study of Discharge-Time Hydrographs in Pipes, Proc. of the First Int. Seminar on Urban Drainage Systems, University of Southampton, England, Sept. 1982.

(240) Saul A.J. & Murrell M.D. (1986) Design Considerations and Maintenance of Storm Sewage Overflows with Particular Emphasis on Storage, Paper presented at the Symp. on Developments in Storm Sewerage Management, 24-25th March 1986, Sheffield City Polytechnic, England.

(241) Saul A.J. & Thornton R.C. (1989) Storage Tanks as a Future Strategy for the Retention of First Flush Load, Paper presented to the Conf. on Drainage and Waste Management into the 1990s, Stakis Earl Grey Hotel, Dundee, 9-11th May 1989.

(242) Saul A.J., Murrell M. & Delo E.A. (1984) Storm Overflow Design with the Aid of a Hydraulic Scale Model, Proc. of the Third Int. Conf. on Urban Storm Drainage, Chalmers University of Technology, Goteborg, Sweden, 4-8th June 1984.

(243) Savini J. & Kammerer J.C. (1961) Urban Growth and the Water Regimen, US Geol. Survey, Water-Supply, Paper 1591-A, pp 43.

(244) Schilling W. (1984) Short Term Flow Forecasting for Real Time Stormwater Control, Proc. of Third Int. Conf. on Urban Storm Drainage, Chalmers University of Technology, Goteborg, Sweden, 4-8th June 1984.

(245) Sharpe D.E. & Kirkbride T.W. (1959) The Operation and Design of a Stilling Pond, Proc. of Inst. of Civil Engineers, Volume 13, August 1959, and Discussions in Volume 16, pp 283, July 1960.

(246) Shearman R.J. (1977) The Speed and Direction of Storm Rainfall Patterns with Reference to Urban Storm Sewer Design, Hydrol. Science Bulletin, Volume 22, pp 421-431.

(247) Shelly P.E. & Kirkpatrick G.A. (1975) Sewer Flow Measurement, A State-of-the-Art Assessment, USEPA Report, EPA-600/2-75-027, NTIS PB-250 371, Nov. 1975, USA.

(248) Shepherd K. (1979) Some Applications of Flood Routing Methods with Particular Reference to Small Streams, PhD Thesis (unpublished), Kingston Polytechnic, Surrey.

(249) Sieker F. (1977) Simulation of Design Storms with Probable Distributions in Time and Space for Storm Drainage Systems, Progr. in Water Technology, Volume 9, pp 509-519.

(250) Sjoberg A. (1976) Calculation of Unsteady Flows in Regulated Rivers and Storm Sewer Systems, No. 87, Chalmers University of Technology, Goteborg, Sweden.

(251) Smisson B.P.M. (1967) Design, Construction and Performance of Vortex Overflows, Symposium of Storm Sewage Overflows, ICE, London.

(252) Smisson B.P.M. (1987) Flow Control in Urban Drainage - The Borehamwood Catchment Plan, Paper presented to the Seminar on Flow Control in Urban Drainage Systems, Hydraulic Research Ltd., Wallingford, England, 26th Nov. 1987.

(253) Smith A.A. (1980) A Generalised Approach to Kinematic Flood Routing, Journal of Hydrological, Volume 45, pp 71-89.

(254) Snyder F.F. (1970) Discussion of Urban Runoff by Road Research Laboratory Method, Reported by Terstriep M.L. & Stall J.B., Journal of the Hydraulics Division, ASCE, Volume 96, No. HY7, Proc. Paper 7384, July 1970, pp 1626-1629.

(255) Stanbridge H.H. (1976) History of Sewage Treatment in Britain, Volume 1, Inst. of Water Pollution Control.

(256) Stickler G.J. (1986) The Dangers of Force Fitting, Paper presented at the WaPUG Autumn Meeting, Council Chambers, Glasgow, 6th Nov. 1986.

(257) Stoker J.J. (1957) Water Waves, Interscience Publications, New York.

(258) Stoneham S. & Kidd C.H.R. (1977) Prediction of Runoff Volume from Fully Sewered Catchments, Report No. 41, Sept. 1977, Institute of hydrology, Wallingford.

(259) Strangeways I.C. & Templeman R.F. (1974) Logging River Level on Magnetic Tape, Paper presented in Water and Water Eng., Volume 178, No. 928.

(260) Styles P.W. & Hedderly H.M. (1982) Sewers and Mains - A Standard Record System, Paper 16 in Sewer Records and Monitoring Flows, Restoration of Sewerage Systems, ICE, London.

(261) Styles P.W. & Robinson G.K. (1984) Planning and Implementing a Sewerage Strategy, Proc. of Int. Conf. on the Planning, Construction, Maintenance and Operation of Sewerage Systems, University of Reading, 12-14th Sept. 1984.

(262) Taylor A.J. & Knott G.E. (1986) Storage Tanks and Sewerage Upgrading, Paper presented at the Symp. on Developments in Storm Sewerage Management, 24-25th March 1986, Sheffield City Polytechnic, England.

(263) Terstriep M.L. & Stall J.B. (1969) Urban Runoff by Road Research Laboratory Method, Journal of the Hydraulics Division, ASCE, Volume 95, No. HY6, Proc. Paper 6878, Nov. 1969, pp 1809-1834.

(264) Terstriep M.L. & Stall J.B. (1974) The Illinois Urban Drainage Area Simulator ILLUDAS, Illinois State Water Survey, Dept. of Registration and Education, Urbana, State of Illinois, Bulletin 58, ISWS-74-BUL58.

(265) Tholin A.L. & Keifer C.J. (1960) Hydrology of Urban Runoff, Trans. Amer. Soc. Civil Engrs., Volume 125, pp 1308-1379.

(266) Thompson J.L. & Lupton A.R.R. (1978) A Method of Assessment of Piped Drainage Systems Taking Account of Surcharge and Overground Flooding, Proc. of the First Int. Conf. on Urban Storm Drainage, University of Southampton, England, April 1978.

(267) Thornton R.C. & Saul A.J. (1986) Some Quality Characteristics of Combined Sewer Flows, Journal of Public Health Engineers, July 1986.

(268) Todd D.K. (1959) Groundwater Hydrology, First Edition, pp 336, Wiley, New York.

(269) Torno H.C. (1975) A Model for Assessing Impact of Storm-Water Runoff and Combined Sewer Overflows and Evaluating Pollution Abatment Alternatives, Water Resource, Volume 9, pp 813-815.

(270) UNESCO (1979) Impact of Urbanisation and Industrialisation on Water Resources Planning and Management, United Nations Eduational, Scientific and Cultural Organisation Studies and Reports in Hydrology, pp 111, Report No. 26.

(271) Verwey A. (1970) Mathematical Model for Flow in Rivers with Realistic Bed Configurations, Int. Courses in Hydraulic and Sanitary Engineering, Report Series No. 12, Delft, Netherlands.

(272) Verworn W. (1978) Electronic Data Collecting System, Proc. of the First Int. Conf. on Urban Storm Drainage, University of Southampton, England, April 1978.

(273) WAA/WRc (1984) Sewerage Rehabilitation Manual, First Edition, April 1984, Water Research Centre and Water Authorities Association.

(274) Walters G.A. (1982) Optimising the Plan Layout of Storm Sewer Networks, Proc. of First Int. Seminar on Urban Drainage System, Sept. 1982, Southampton.

(275) Warrilow D.A. (1980) A Study of Storm Profiles in the Meteorological Office Flood Studies Computer Program Suite, Met. Off. 8, Rainfall Memorandum No. 19 (unpublished document), August 1980.

(276) Watkins D.C. (1987) The Hydraulic Design of Soakways, Paper presented at the Seminar on Flow Control in Urban Drainage Systems, 26th Nov. 1987, HRS, Wallingford, England.

(277) Watkins L.H. (1962) The Design of Urban Storm Sewers, Road Research Laboratory Technical Paper, No. 55, Dept. of Scientific and Industrial Research, HMSO, London.

(278) Watkins L.H. (1966) Runoff from Combined Rural and Urban Areas, Chapter 7 of Thorn R.B. (ed), River Engineering and Water Conversation Works, pp 111-121, Butterworths, London.

(279) Watkins L.H. (1970) Discussion of Urban Runoff by Road Research Laboratory Method, Reported by Terstriep M.L. & Stall J.B., Journal of the Hydraulics Division, ASCE, Volume 96, No. HY7, Proc. Paper 7384, July 1970, pp 1625-1626.

(280) Watts I.L. (1986) Monitoring and Analysis, Sewerage : the Practical Options, Paper presented in the Public Health Engineer, Volume 14, No. 1, Jan. 1986.

(281) Watts I.L. (1986) Sewerage : The Practical Options, 1. Monitoring and Analysis, the Public Health Engineer, Volume 14, No. 1, January 1986.

(282) Wilcox P. (1985) Discussions on Detectronic Flow Loggers, Reported by Ashley R.M., Internal Paper, Dundee Institute of Technology.

(283) Williams D.W. (1984) Hydraulic Analysis - The Role of Sewer Flow Surveys, Ext. Report No. 136E, Water Res. Centre (WRc), April 1984.

(284) Williams D.W. (1984[1]) Checking on the WASSP-Sim Model Using Flow Measurements, Proc. of Seminar on the Wallingford Procedure, June 1984, Dundee Inst. of Technology.

(285) Williams D.W. (1985) Discussion Document to the Guide to Sewer Flow Surveys, Unpublished Document, Water Research Centre.

(286) Williams D.W. & Bartlett J.M. (1984) Application of Hydraulic Analysis Techniques to Investigate an Overloaded Sewerage System, Proc. of Third Int. Conf. on Urban Storm Drainage, 4-8th June 1984, Goteborg, Sweden,

(287) Wilson E.M. (1983) Engineering Hydrology, Third Edition, MacMillan Civil Engineering Hydraulics.

(288) Wilson G. Day Level Drainage in Eighteen Century, Fife Coal Mines, Sources Unknown.

(289) Woolhiser D.A. & Liggett J.A. (1967) Unsteady One-Dimensional Flow over a Plane -- The Rising Hydrograph, Water Resources Research, Volume 3, No. 3, June 1967, pp 753-771.

(290) WRc (1987) A Guide to Short Term Flow Surveys of Sewer Systems, Water Research Centre, Swindon.

(291) WRc/WAA (1981) Sewers for Adoption, Water Research Centre and Water Authorities Association, First Edition.

(292) WRc/WAA (1985) Sewers for Adoption : A Design and Construction Guide for Developers, Second Edition, Water Research Centre and Water Authorities Association.

(293) WRc/WAA (1986) Sewerage Rehabilitation Manual, Second Edition, June 1986, Water Research Centre and Water Authorities Association.

(294) Yen B.C. (1973) Open-Channel Flow Equations Revisited, Journal of Eng. Mech. Division, ASCE, Volume 99, No. EM5, pp 979-1009, October 1973.

(295) Yen B.C. (1986) Rainfall-Runoff Process on Urban Catchment and its Modelling, Proc. of Urban Drainage Modelling, Int. Symp. on Comparison of Urban Drainage Models with Real Catchment Data, 8-11 April 1986, UDM '86, Dubrovnik, Yugoslavia.

(296) Yen B.C. & Chow V.T. (1969) A Laboratory Study of Surface Runoff due to Moving Rainstorms, Water Resources Res., Volume 5, pp 989-1006.

(297) Yen B.C. & Sevuk A.S. (1975) Design of Storm Sewer Networks, Proc. of ASCE, Journal of Environ. Engng. Division, Volume 101 (EE4), pp 535-553.

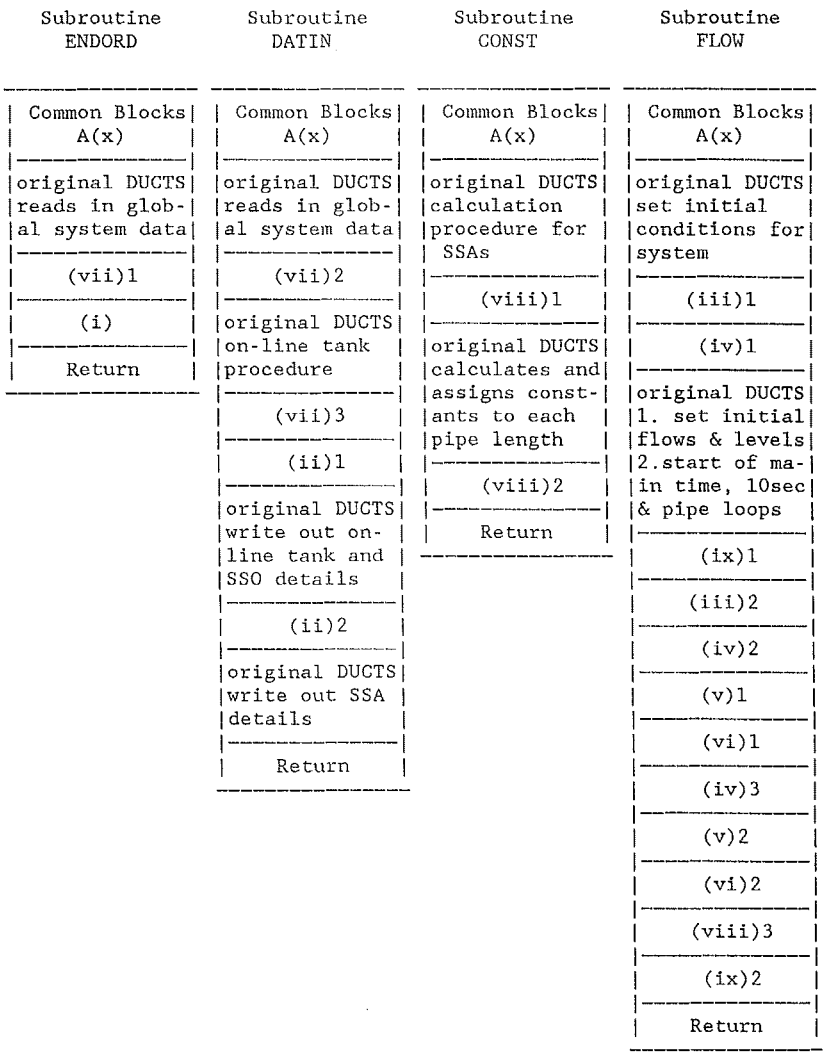
(298) Yevjevich V. & Barnes A.H. (1970) Flood Routing through Storm Drains, Part IV, Numerical Computer Methods of Solution, Colorado State University, Hydrol. Paper No. 46, pp 47.

(299) Yu Y.S. & McNown J.S. (1964) Runoff from Impervious Surfaces, University of Kansas, Report to Waterways Experiment Station, Corp. of Engineer, Vicksburgh, Mississippi.

(300) Zaghloul N.A. (1977) Evaluation of Storm Water Routeing Models, Proc. Instn. Civil Engrs., Part 2, Volume 63, December 1977, pp 925-933.

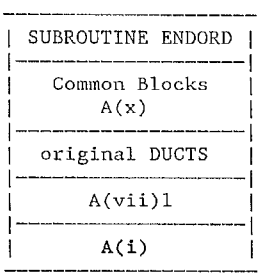
APPENDIX A
PRESENTATION OF COMPUTER PROGRAM

Appendices A(i) to A(x) give details of the programming required to effect modifications to DUCTS in the development of the parallel pipe model DUPPERS. Each section includes a diagram showing the relative position of the programming components. The following figure shows the overall locations of these sections. Subroutines in the following figure are in the same sequence as in DUPPERS.



APPENDIX A(i)
END ORDER AND MDOWN NUMBER COMPUTATION

[SUBROUTINE ENDORD]



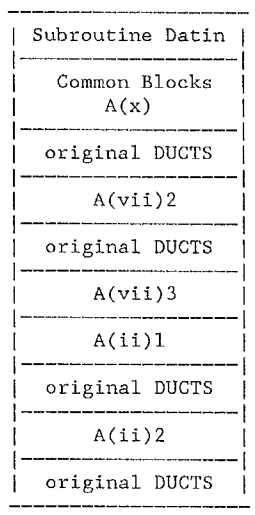

```
C   START OF ROUTINE TO DETERMINE END ORDER & MANHOLE DOWNSTREAM
C   PARALLEL PIPES 500 AND 600 ARE TREATED AS MAIN PIPE 1.000 AS
C   IN THE DENTRITIC SYSTEMS
C   NS IS THE LINE JUST BEFORE PIPE 1.000
100     CONTINUE
        J=0
        LP=2
        LZ=0
        LB=0
        LC=0
        LD=1
        IB1=IBCODE(1)
        IP1=IPCODE(1)
        NENDO(1)=1
        DO 170 I=2,N+1
        IB3=IBCODE(I)
        IP3=IPCODE(I)
        IF(IB3.LT.500)GOTO 50
        IF(IB3.NE.800)GOTO 48
        IB2=80
        IP3=0
        GOTO 55
48      IB2=IB3-(IB3-1)
        GOTO 55
50      IB2=IB3
55      IP2=IP3
        IF(IB2.NE.IB1)GOTO110
C PIPE IB2 IS DIRECTLY DOWN FROM IB1
        MDNEW(I-1)=I
        GOTO160
110     IF(IB2.LT.IB1)GOTO120
C LOCATION IS NOW ON ANOTHER BRANCH AT ITS HEAD
        IF(IP2.NE.0) CALL ERROR(I2,3)
        NST=NST+1
        NSTACK(NST)=I-1
        LSTACK(NST)=IB1
        GOTO160
C LOCATION IS NOW BELOW A JUNCTION, SO NEXT VALUE IS DOWNSTREAM
120     CONTINUE
```

```
                                                                      A(i)
C       IF (IP2.EQ.0) CALL ERROR(I2,4)
C TOP STACK VALUE AND IB1 FLOW INTO IB2
        JS=NST
130     IQ=LSTACK(JS)
        IF(IQ.EQ.IB2)GOTO140
        JS=JS-1
        GOTO130
C ALL PIPES BETWEEN JS & NST IN THE STACK JOIN AT IB2
140      DO 150 IY=JS , NST
        II=NSTACK(IY)
        MDNEW(II)=I
        IF(IY.EQ.NST)GOTO150
        III=LSTACK(IY+1)
150     CONTINUE
        MDNEW(I-1)=I
        NST=JS-1
160     CONTINUE
        NENDO(I)=I
        IB1=IB2
        IP1=IP2
        IF(IB3.EQ.700.AND.IP3.GT.1)GOTO 56
        IF(IB3.EQ.ICPODE(LP).AND.IP3.EQ.IACODE(LP))GOTO 80
        IF(NANI(I).EQ.6)GOTO 17
        IF(IB3.EQ.ISCODE(LD).AND.IP3.EQ.ITCODE(LD))GOTO 18
        GOTO 170
56      J=J+1
        LDOWN(J)=MDNEW(I-1)
        GOTO 170
80      LZ=LZ+1
        IAN(LZ)=MDNEW(I-1)
        LP=LP+1
        GOTO 170
18      LC=LC+1
        KON(LC)=MDNEW(I-1)
        LD=LD+1
        GOTO 170
17      LB=LB+1
        LON(LB)=MDNEW(I-1)
        IF(ISCODE(LD).NE.0)GOTO 170
        LC=LC+1
        KON(LC)=0
        LD=LD+1
170     CONTINUE
        DO 1 IT=1,J
1       WRITE(5,1099)LDOWN(IT)
1099    FORMAT(1H ,'  M/DOWN FOR CC IS ',I4)
        DO 2 ITT=1,LZ
2       WRITE(5,1098)IAN(ITT)
1098    FORMAT(1H ,'  IAN(I) FOR CC EXP. 0 IS ',I3)
C
        DO 4 IAT=1,LB
4       WRITE(5,1096)LON(IAT),KON(IAT)
1096    FORMAT(//,1X,'     LON(I)= ',I3,'     KON(I)= ',I3)
C
        JY=1
        JL=1
        JU=2
        DO 3 I=1,N
```

```
                                                                      A(i)
      IF(IBCODE(I).EQ.IUB(JL).AND.IPCODE(I).EQ.IUP(JL))GOTO 81
      IF(IBCODE(I).EQ.ICPODE(JU).AND.IPCODE(I).EQ.(IACODE(JU)-1))
     1GOTO 82
      MDOWN(I)=MDNEW(I)
      GOTO 3
81    MDOWN(I)=LDOWN(JL)
      JL=JL+1
      GOTO 3
82    MDOWN(I)=IAN(JY)
      JY=JY+1
      JU=JU+1
3     CONTINUE
      DO 6 IN=1,N
6     WRITE(5,1097)MDOWN(IN)
1097  FORMAT(1H ,'           M/H DOWNSTREAM = ',I3)
      DO 8 KQ=1,N
8     WRITE(5,988)MDNEW(KQ)
C
C   NUMBERING PROCEDURE FOR PARALLEL PIPES ENDS HERE
C ***************************************************************************
```

APPENDIX A(ii)
PROCEDURE OF SYSTEM DATA INPUT AND OUTPUT

[SUBROUTINE DATIN]



Appendix [A(ii)1]

```
C   DWF DETERMINATION FOR DENTRITIC AND PARALLEL PIPES SYSTEMS
        DO 631 I=1,N
        DWF(I)=0.0
631     CONTINUE
        TDWF=0.0
        MA=1
        MY=1
        MB=0
        DO 40 I=1,N
        IF(IBCODE(I).EQ.ILAST(MY).AND.IPCODE(I).EQ.ILAPE(MY))GOTO 83
        GOTO 88
83      MY=MY+1
        GOTO 40
88      IF(IBCODE(I).LE.500.OR.IBCODE(I).GE.700)GOTO 4
        IF(IBCODE(I).GT.500.AND.IPCODE(I).EQ.0)GOTO 64
        GOTO 4
64      TDWF=0.0
        DWF(I)=0.0
4       IF(ASP(I).LT.0.00001) ASP(I)=DWFG*XL(I)
        MD=MDOWN(I)
        TDWF=ASP(I)/1000.
        DWF(I)=TDWF+DWF(I)
        DWF(MD)=DWF(MD)+DWF(I)
        TDWF=DWF(I)*1000.
C DWF(I) IS THE ACCUMULATED DWF (cumecs) DOWN TO PIPE I
        IF(NANI(I).EQ.2) GOTO 35
        IF(NANI(I).EQ.4) GOTO 33
```

```
      IF(IBCODE(I).NE.800)GOTO 640
C
C   PIPE GRADIENT COMPUTATION INCLUDING BRIDGING PIPES
      MB=MB+1
      SL(I)=(YINV(I)-GL(I))/XL(I)
      SLB(MB)=SL(I)
      GOTO 650
640   CONTINUE
      SL(I)=(YINV(I)-YINV(MD))/XL(I)
650   CONTINUE
      IF(IBCODE(I).EQ.700)GOTO 84
      IF(IBCODE(I).EQ.800)GOTO 85
      IF(NANI(I).EQ.6) GOTO 32
      IF(IBCODE(I).EQ.ICPODE(MA).AND.IPCODE(I).EQ.IACODE(MA))GOTO401
C
C   WRITE OUT SINGLE PIPE DETAILS IN CHECK FILE -- DUPCHK.DAT
      WRITE(I2,125)IBCODE(I),IPCODE(I),XL(I),YINV(I),SL(I),
     1D1(I),D2(I),XKS(I),CA(I),AIP(I),PRP(I),FAP(I),
     2TDWF,GL(I),PAPG(I),EM(I)
      GOTO 402
C
C   PARALLEL PIPES DETAILS WRITE OUT IN CHECK FILE
401   WRITE(I2,170)IBCODE(I),IPCODE(I),XL(I),YINV(I),SL(I),
     1D1(I),D2(I),XKS(I),CA(I),AIP(I),PRP(I),FAP(I),
     2TDWF,GL(I),PAPG(I),EM(I)
      MA=MA+1
402   IF(SL(I).GT.0) GOTO 25
      CALL ERROR(I2,12)
      WRITE(I2,270)I
25    CONTINUE
      GOTO 40
C   CROSS-CONNECTION BRANCH & PIPE NUMBERS
84    WRITE(I2,320)IBCODE(I),IPCODE(I)
      GOTO 40
C   BRIDGING PIPE BRANCH & PIPE NUMBERS
85    WRITE(I2,325)IBCODE(I),IPCODE(I)
      GOTO 40
32    WRITE(I2,323)IBCODE(I),IPCODE(I),XL(I),YINV(I),SL(I)
      NANI(I)=3
      GOTO 40
33    WRITE(I2,128) IBCODE(I),IPCODE(I)
      GOTO 40
35    SL(I)=YINV(I)
      WRITE(I2,126) IBCODE(I),IPCODE(I)
      NSSA=NSSA+1
      isp(i)=NSSA
      IF(NSSA.GT.150) CALL ERROR(I2,11)
40    CONTINUE
      pimp=pvarea*100./at
      WRITE(I2,275)AT
      AT=AT/100.
C     AT IS TOTAL CATCHMENT AREA IN SQKM
      WRITE(I2,127)
      WRITE(I2,129)
      WRITE(I2,405)
      WRITE(I2,410)
      WRITE(I2,415)
      WRITE(I2,418)
```

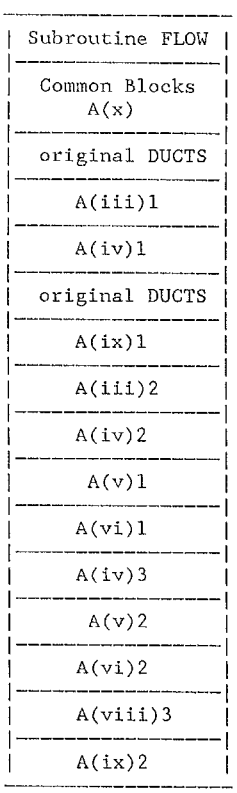
```
      WRITE(I2,230)
C     NOW WRITE OUT THE CROSS-CONNECTION AND
C     BRIDGING PIPE DETAILS IN FULL
      IF(IG.LE.0)GOTO 420
      WRITE(I2,505)
      WRITE(I2,510)
      WRITE(I2,515)IG
      WRITE(I2,520)
      WRITE(I2,525)
      WRITE(I2,530)
      WRITE(I2,535)
      WRITE(I2,540)
      LW=0
      DO 425 I=1,N
      IF(IBCODE(I).EQ.700)GOTO 430
      IF(IBCODE(I).EQ.800)GOTO 434
      GOTO 425
430   LW=LW+1
      MORE(LW)=MDOWN(I)
      GOTO 425
434   MORY(LW)=MDOWN(I)
425   CONTINUE
      DO 435 I=1,IG
435   WRITE(I2,545)ICCODE(I),IRCODE(I),ICNODE(I),IRCODE(I),MORE(I)
     1,MORY(I),IWT(I),WL(I),GCO(I),YINC(I),D3(I),IQSET(I),CD(I)
      WRITE(I2,550)
      WRITE(I2,555)
      WRITE(I2,560)
      WRITE(I2,565)
      WRITE(I2,570)
      DO 440 I=1,IG
440   WRITE(I2,575)ICNODE(I),IRCODE(I),ICPODE(I),IACODE(I),XP(I)
     1,YCC(I),GCC(I),D4(I),SLB(I)
      WRITE(I2,231)
      WRITE(I2,230)
      GOTO 445
420   WRITE(I2,500)
445   CONTINUE
```

Appendix [A(ii)2]

```
C  NOW WRITE OUT OUTFALLL DETAILS
      DO 50 J=1,5
50    IOUT(J)=500
      WRITE(I2,198)
      WRITE(I2,201)
      WRITE(I2,202)IX
      WRITE(I2,204)
      DO 433 I=1,IX
433   WRITE(I2,203)ILAST(I),ILAPE(I),YINT(I)
      WRITE(I2,231)
      WRITE(I2,230)
C
C     END
```

APPENDIX A(iii)
FREE SURFACE OVERFLOW COMPUTATION

[SUBROUTINE FLOW]



Appendix [A(iii)1]

```
C     PHYSICAL DIMENSIONS AND OTHER DETAILS
C     READ-IN FROM STORED ARRAYS
        DO 27 LC=1,IG
        MRE=MORE(LC)
        MRY=MORY(LC)
        STINV=YINV(MRE)
        CRINV=YINV(MRY)
        STDIA=FLOAT(D1(MRE))/1000.0
        CRDIA=FLOAT(D1(MRY))/1000.0
        STIL(LC)=STINV
        CRIL(LC)=CRINV
        STSL(LC)=STINV+STDIA
        CRSL(LC)=CRINV+CRDIA
C
```

```
        CNDIA=FLOAT(D4(LC))/1000.0
        CNSL(LC)=CNDIA+YCC(LC)
        BNSL(LC)=CNDIA+GCG(LC)
        CREHGT=FLOAT(D3(LC))/1000.0
        YAVE=(YINC(LC)+STINV)/2.0
C       WLEV(LC)=YCC(LC)+CREHGT
        WLEV(LC)=YAVE+CREHGT
        ARAF(LC)=0.7853981*STDIA*STDIA
        ARAS(LC)=0.7853981*CRDIA*CRDIA
        ARAN(LC)=0.7853981*CNDIA*CNDIA
27      CONTINUE
```

Appendix [A(iii)2]

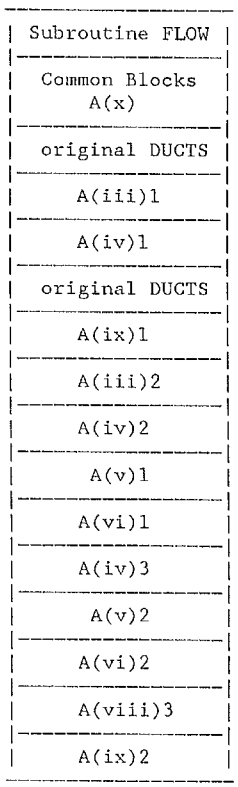
```
C ---------------------------------------
C      CROSS CONNECTION OVERFLOW  (VR. 3)
C       Major update on Jan 1989
C       Reverse flow update on feb 89
C ---------------------------------------
C       IF(IBCODE(J).NE.700)GOTO 610
C   Cross connection physical & hydraulic details
C   ---------------------------------------------
611     CONTINUE
        ISTACK=0
        NF=NF+1
        MDD=MORY(NF)
        MDE=MDD-1
        YLV=YN(J)
        NNF=MORE(NF)
        QFBF=QFB(NNF)
        QFBS=QFB(MDD)
        QFBB=QFB(MDE)
        TD1=FLOAT(D1(NNF))/1000.0
        STDIA=STSL(NF)-STIL(NF)
        CRDIA=CRSL(NF)-CRIL(NF)
        IDD1=D4(NF)
        TD2=(FLOAT(IDD1))/1000.0
        CNDIA=CNSL(NF)-YCC(NF)
        ID2=D3(NF)
        DD1=(FLOAT(ID2))/1000.0
        CSAF=ARAF(NF)
        CSAS=ARAS(NF)
        CSAB=ARAN(NF)
        YCOMP=DD1-TD1
C
        QFIN=QW(J)
        QFST=QFIN
        QSTR=QCOLD(J)
        QCRO=QOVE(J)
        QRMF=QFIN
        QRMS=QSIN
        VCOLD=VCC(J)
C   Level determination in C.C. manhole
C   -----------------------------------
754     CONTINUE
        VCOLD=VCOLD+((QFIN-QSTR-QCRO)*10.0)
```

```
      IF(VCOLD.LE.0.0)GOTO 782
      YCO=VCOLD/2.4
      IF(YCO.GT.DD1)GOTO 759
C   Determine the outflow and overflow directions
C   ---------------------------------------------
760   CONTINUE
      QCRO=0.0
      QSTR=QFIN
      GOTO 640
782   CONTINUE
      VCOLD=VCOLD+((QFIN-QSTR)*10.0)
      IF(VCOLD.LE.0.0)VCOLD=0.0
      YCO=VCOLD/2.4
      IF(YCO.LE.0.0)GOTO 760
      IF(YCO.GT.DD1)GOTO 759
      PRDEP=YCO/TD1
      KKL=0
783   CONTINUE
      KKL=KKL+1
      DDD=DPROP(NDKS(J+1),KKL)
      IF(DDD.LT.PRDEP)GOTO 783
      QSTR=QFBF*FLOAT(KKL)/1000.0
      QCRO=0.0
      GOTO 756
C   Determine the overflow and straight flow
C   ----------------------------------------
759   CONTINUE
      IF(YCO.LE.DD1)GOTO 780
      YHW=YCO-DD1
      YHAV=0.083*(YHW/DD1)
      CD1=0.602+YHAV
      IF(IWT(NF).NE.0)CD1=1.204+YHAV
C     WRITE(5,1989)CD1
C1989 FORMAT(/,1X,'COEFFICIENT (CD1) = ',F10.5,/)
      YCHD=YHW**1.5
      QCRO=CD1*WL(NF)*3.13*YCHD
      IF(QCRO.LE.QFBB)GOTO 758
      YBEN=YCO-(0.5*TD2)
      QCRO=0.6*CSAB*SQRT(19.62*YBEN)
758   CONTINUE
      PRDEP=YCO/TD1
      IF(PRDEP.GE.1.0)GOTO 755
      KKK=0
788   CONTINUE
      KKK=KKK+1
      DDD=DPROP(NDKS(J+1),KKK)
      IF(DDD.LT.PRDEP)GOTO 788
      QSTR=QFBF*FLOAT(KKK)/1000.0
      GOTO 781
755   CONTINUE
      YCEN=YCO-(0.5*TD1)
      IF(YCEN.LE.0.0)YCEN=0.0
      QSTR=QFIN-QCRO
781   CONTINUE
      QCOUT=QSTR+QCRO
      IF(ABS(QFIN-QCOUT).LE.0.001)GOTO 780
      IF(QCOUT.GT.QFIN)YCO=0.99*YCO
      IF(QCOUT.LT.QFIN)YCO=1.01*YCO
```

```
      GOTO 759
780   CONTINUE
      QSIN=QSIN+QCRO
      GOTO 297
C
C   FREE SURFACE PROCEDURE ENDS
```

APPENDIX A(iv)
PRESSURISED OVERFLOW COMPUTATION

[SUBROUTINE FLOW]



Appendix [A(iv)1]

```
YLVF=YCO
YRLF=YCO+STIL(NF)
YRLS=YSW+CRIL(NF)
```

Appendix [A(iv)2]

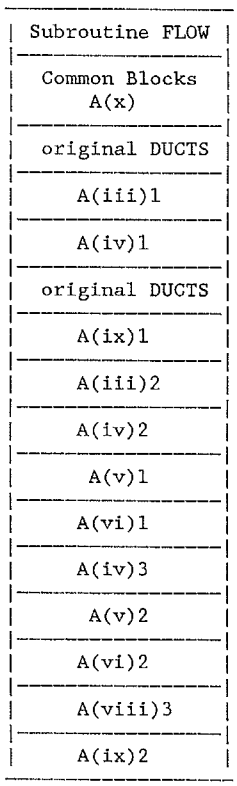
```
C   PRESSURISED OVERFLOW PROCEDURES
      IF(YRLS.GT.ID2)GOTO 40
```

Appendix [A(iv)3]

```
40      YDIFF=YRLF-YRLS
        QCRO=0.59*ARAN(NF)*(SQRT(19.62*YDIFF))
        QCONF=QRMF-QCRO
        QCONS=QRMS+QCRO
        YNN=(QCONF*TSTEP)/AMHF
        YNNS=(QCONS*TSTEP)/6.6
        IF(YNN.GT.STSL(NF))GOTO 91
        Q2=QCONF
        GOTO 92
91      CONTINUE
        SRTF=SQRT(19.62*(YNN-(STDIA/2.0)))
        Q2=0.59*ARAF(NF)*SRTF
92      IF(YNNS.GT.CRSL(NF))GOTO 93
        QW(MRY)=QW(MRY)+QCONS
        GOTO 297
93      CONTINUE
        SRTS=SQRT(19.62*(YNNS-(STDIA/2.0)))
        QW(MRY)=QW(MRY)+(0.65*ARAS(NF)*SRTS)
        GOTO 297
C
C    SURCHARGED FLOW ENDS
```

APPENDIX A(v)
HEAD BALANCE & FLOW COMPUTATION

[SUBROUTINE FLOW]



Appendix [A(v)1]

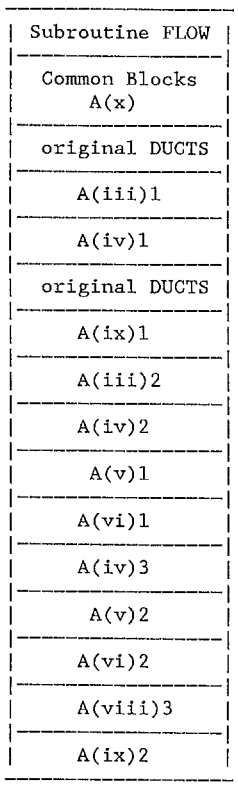
```
C   HEAD BALANCE BEHAVIOUR IN CROSS-CONNECTION
C   NO CROSS FLOW OR REVERSE FLOW IN BRIDGING PIPE
      IF(YRLS.EQ.YRLF)GOTO 35
```

Appendix [A(v)2]

```
35      CONTINUE
        Q2=QRMF
        QCRO=0.0
        QREV=0.0
        YNN=YCO
        QFIN=QRMF
        QSIN=QRMS
        QW(MRY)=QW(MRY)+QRMS
        GOTO 297
C
C   HEAD BALANCE PROCEDURES END
```

APPENDIX A(vi)
REVERSE FLOW COMPUTATION

[SUBROUTINE FLOW]



Appendix [A(vi)1]

```
C     REVERSE FLOW PROCEDURE
      IF(YRLS.GT.YRLF)GOTO 34
```

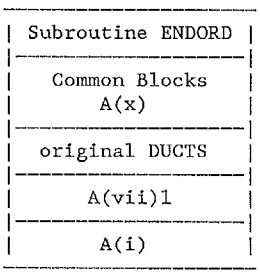
Appendix [A(vi)2]

```
C
34    CONTINUE
      IF(YRLF.LE.ID2)GOTO 95
      IF(YRLS.GE.CNSL(NF))GOTO 47
      YHEAD=(0.5*VELS*VELS)/19.62
      GOTO 48
47    YHEAD=(VELS*VELS)/19.62
```

```
48      YDIFF=YRLS-YRLF-YHEAD
        IF(YDIFF.LE.0.0)GOTO 42
        QREV=0.59*ARAN(NF)*(SQRT(19.62*YDIFF))
        QRMF=QRMF+QREV
        QRMS=QRMS-QREV
        YNN=(QRMF*TSTEP)/AMHF
        YNNS=(QRMS*TSTEP)/6.6
        IF(YNN.GT.STSL(NF))GOTO 98
        Q2=QCONF
        GOTO 110
98      CONTINUE
        SRTF=SQRT(19.62*(YNN-(STDIA/2.0)))
        Q2=0.59*ARAF(NF)*SRTF
110     IF(YNNS.GT.CRSL(NF))GOTO 111
        QW(MRY)=QW(MRY)-QREV
        GOTO 297
111     CONTINUE
        SRTS=SQRT(19.62*(YNNS-(STDIA/2.0)))
        QW(MRY)=QW(MRY)+(0.65*ARAS(NF)*SRTS)
        GOTO 297
C
95      IF(ID2.GE.CNSL(NF))GOTO 49
        YHEAD=(VELS*VELS)/19.62
        GOTO 51
49      YHEAD=(1.5*VELS*VELS)/19.62
51      YDIFF=YRLS-ID2-YHEAD
        IF(YDIFF.LE.0.0)GOTO 42
        QREV=1.705*CD1*WL(NF)*(YDIFF**1.5)
        QRMF=QRMF+QREV
        QRMS=QRMS-QREV
        YNN=(QRMF*TSTEP)/AMHF
        YNNS=(QRMS*TSTEP)/6.6
        IF(YNN.GT.STSL(NF))GOTO 112
        Q2=QCONF
        GOTO 113
112     CONTINUE
        SRTF=SQRT(19.62*(YNN-(STDIA/2.0)))
        Q2=0.59*ARAF(NF)*SRTF
113     IF(YNNS.GT.CRSL(NF))GOTO 115
        QW(MRY)=QW(MRY)-QREV
        GOTO 297
115     CONTINUE
        SRTS=SQRT(19.62*(YNNS-(STDIA/2.0)))
        QW(MRY)=QW(MRY)+(0.65*ARAS(NF)*SRTS)
        GOTO 297
C
C   REVERSE FLOW COMPUTATION ENDS
```

APPENDIX A(vii)
SYSTEM DATA READ-IN PROCEDURE

[SUBROUTINE ENDORD - A(ii)1]
[SUBROUTINE DATIN - A(ii)2 & A(ii)3]



Appendix [A(vii)1]

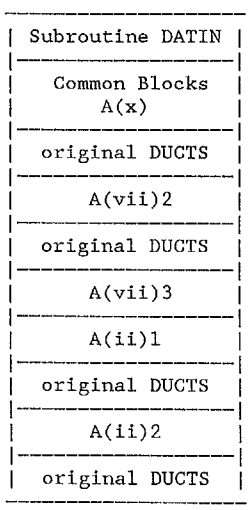
```
C     INITIAL BRANCH & PIPE DATA READ-IN FOR
C     END ORDER & DOWNSTREAM MANHOLE NUMBER DETERMINATION
          N=0
          MC=0
          NB=0
          NF=0
          NQ=0
          NN=1
10        IF(N.GT.(men-2))CALL ERROR(I2,500)
          READ(I3,1000,ERR=11)IB,IP,IA
          IF(IB.EQ.ICPODE(NN).AND.IP.EQ.IACODE(NN))GOTO 15
          GOTO 12
15        N=N+1
          IBCODE(N)=ICNODE(NN)
          IPCODE(N)=IRCODE(NN)
          NANI(N)=0
          NN=NN+1
          GOTO 12
11        IX=2
          WRITE(I2,500)IX,N
          CALL ERROR(I2,2)
12        CONTINUE
20        IF(IB.LT.0)GOTO40
C
C     PIPE SORTING
          IF(IB-700)13,610,25
C
C     NORMAL PIPES
13        CONTINUE
          N=N+1
          IBCODE(N)=IB
          IPCODE(N)=IP
          NANI(N)=IA
          IF(IA.EQ.4) IOVF=2
          WRITE(5,830)IB,IP
```

```
830     FORMAT(1X,'ITS NOW BRANCH NO. ',I3,',  PIPE NO. ',I3)
        IF(IA.EQ.6)GOTO 7
30      GOTO10
7       NQ=NQ+1
        READ(I3,1000,ERR=11)IB,IP,IA
        IST(NQ)=IB
        ISCODE(NQ)=IP
        ITCODE(NQ)=IA
        GOTO 10
C
C       CROSS-CONNECTION OVERFLOW
610     MC=MC+1
        N=N+1
        ICCODE(MC)=IB
        IRCODE(MC)=IP
        NANI(N)=IA
        IBCODE(N)=IB
        IPCODE(N)=IP
        IF(IP.LE.1)GOTO 21
        NG=0
22      NG=NG+1
        IEA=IBCODE(N)-IBCODE(N-NG)
        IF(IEA.GT.180.AND.IEA.LE.200)GOTO 23
        GOTO 22
23      NF=NF+1
        IUB(NF)=IBCODE(N-NG)
        IUP(NF)=IPCODE(N-NG)
21      WRITE(5,840)IP,IA
840     FORMAT(1H ,1X,'ITS NOW AT CROSS CONN. NO. ',I3,', TYPE ',I3)
        GOTO 10
C
C    BRIDGING PIPE
25      ICNODE(MC)=IB
        ICPODE(MC)=IP
        IACODE(MC)=IA
        WRITE(5,850)MC,IP,IA
850     FORMAT(1X,'CONN. PIPE',I2,'; D/S BRANCH ',I4,' PIPE NO ',I3,/)
        GOTO 10
C
C    OUTFALL NUMBERS READ-IN
40      CONTINUE
        NB=NB+1
        READ(I3,1000)IBCODE(N+1),IPCODE(N+1)
        ILAST(NB)=IBCODE(N+1)
        ILAPE(NB)=IPCODE(N+1)
        WRITE(5,880)NB,IBCODE(N+1),IPCODE(N+1)
        IF(IB.EQ.-1)GOTO 667
        N=N+1
        IBCODE(N)=ILAST(NB)
        IPCODE(N)=ILAPE(NB)
        GOTO 10
880     FORMAT(1X,'OUTFALL ',I3,'; BRANCH ',I3,' PIPE ',I3)
C
C END OF INITIAL READING IN OF DATA
C
C    WRITE OUT READ-IN DATA ON VDU FOR IMMEDIATE CHECKS
667     CONTINUE
        DO 111 I=1,3
```

```
111     WRITE(5,1062)
        WRITE(5,800)N+1
800     FORMAT(1H ,'THERE ARE A TOTAL OF ',I3,' PIPES',/)
        WRITE(5,810)MC
810     FORMAT(1H ,'CROSS CONNECTION :-  ',I3,/)
        WRITE(5,820)MC
820     FORMAT(1H ,'BRIDGING PIPE    :-  ',I3,/)
        WRITE(5,860)NB
860     FORMAT(1H ,'FINAL OUTFALL    :-  ',I3,/)
        DO 222 I=1,N
222     WRITE(5,890)IBCODE(I),IPCODE(I)
890     FORMAT(1H ,'BRANCH = ',I4,'  PIPE = ',I3)
        DO 223 I=1,MC
223     WRITE(5,891)ICNODE(I),I,ICPODE(I),IACODE(I)
891     FORMAT(1H ,'CC ',I3,' NO.',I2,'; D/S BRANCH ',I3,' PIPE ',I2)
        DO 225 L=1,NB
225     WRITE(5,892)L,ILAST(L),ILAPE(L)
892     FORMAT(1H ,'OUTFALL ',I2,' ; BRANCH = ',I3,' PIPE = ',I3)
        DO 228 I=1,NF
228     WRITE(5,893)IUB(I),IUP(I)
893     FORMAT(1H ,'BRANCH IUB = ',I3,' PIPE IUP = ',I3)
        DO 112 I=1,3
112     WRITE(5,1062)
C
C     END
```

A(vii)

[SUBROUTINE DATIN]



Appendix [A(vii)2]

```
C   READ IN MAIN PIPE DATA CARD BY CARD
C   SORTS AND WRITES OUT THE STRUCTURE OF THE SYSTEM
C   WATCHING FOR ANCILLARIES
C
        NONLT=0
        IOUTJ=0
        pvarea=0.
        AT=0.
        IG=0
        NS=0
        IX=0
        IE=0
C       DO 10 I=1,N
C       IE=NENDO(I)
300     IE=IE+1
        HLI(IE)=0
20      READ(I3,150,ERR=9000)IBCODE(IE),IPCODE(IE),NANI(IE),L,
     1  GL(IE),YINV(IE),D1(IE),D2(IE),CA(IE),PI(IE),EM(IE),HLI(IE),
     2  XKS(IE),AIP(IE),PRP(IE),FAP(IE),SI(IE),PAPG(IE),ASP(IE)
        IF(IBCODE(IE).EQ.ICPODE(IG).AND.IPCODE(IE).EQ.IACODE(IG))GOTO18
        IF(IBCODE(IE).LT.0)GOTO 305
        IF(NANI(IE).NE.4) GOTO 6
        NOVER=NOVER+1
        iovf=1
        IF(NOVER.GT.10) CALL ERROR(I2,10)
        IOV(IE)=NOVER
        JOV(NOVER)=L
        OVA(NOVER)=CA(IE)+YINV(IE)
        KOV(NOVER)=D2(IE)
```

```
        LOV(NOVER)=D1(IE)
        GOTO 300
C
C   PIPE SORTING FOR READ IN DATA
6       IF(IBCODE(IE)-700)82,80,81
```

Appendix [A(vii)3]

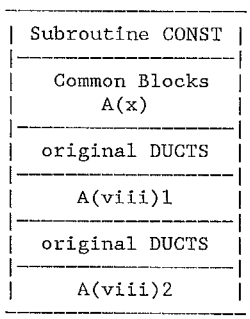
```
C   NORMAL PIPES DATA INCLUDING PARALLEL PIPES 500 & 600
82      CONTINUE
7       IF(XL(IE).GT.0.)GOTO 13
        XL(IE)=FLOAT(L)
13      IF(GL(IE).LT..005) GL(IE)=YINV(IE)+100.
        IF(HLI(IE).EQ.0)HLI(IE)=1
        IF(XKS(IE).EQ.0)XKS(IE)=RH
        IF(AIP(IE).EQ.0)AIP(IE)=IFIX(PIMP)
        IF(PRP(IE).EQ.0)PRP(IE)=GPRP
        IF(FAP(IE).EQ.0)FAP(IE)=GFAP
        IF(SI(IE).EQ.0) SI(IE)=2
        IF(PAPG(IE).EQ.0) PAPG(IE)=-2
        ALPHA(IE)=GALPH
        AT=AT+CA(IE)
        PERPER=FLOAT(AIP(IE)+PRP(IE))
        pvarea=pvarea+ca(ie)*PERPER/100.
10      CONTINUE
        WRITE(5,337)IBCODE(IE),IPCODE(IE)
337     FORMAT(1H ,'BRANCH IN DATIN = ',I3,' PIPE = ',I3)
        GOTO 300
C
C    CROSS-CONNECTION DATA READ-IN
80      IG=IG+1
        ICCODE(IG)=IBCODE(IE)
        IRCODE(IG)=IPCODE(IE)
        NINI(IG)=NANI(IE)
        XL(IE)=FLOAT(L)
        WL(IG)=FLOAT(L)
        GCO(IG)=GL(IE)
        YINC(IG)=YINV(IE)
        D3(IG)=D1(IE)
        IQSET(IG)=D2(IE)
        CD(IG)=CA(IE)
        IWT(IG)=PI(IE)
        WRITE(5,338)ICCODE(IG),IRCODE(IG)
338     FORMAT(1H ,'CC IN DATIN = ',I3,' NO = ',I2)
        GOTO 300
C
C   BRIDGING PIPE DATA READ-IN
81      ICNODE(IG)=IBCODE(IE)
        ICPODE(IG)=IPCODE(IE)
        IACODE(IG)=NANI(IE)
C       XL(IE)=FLOAT(L)
        XP(IG)=FLOAT(L)
        GCC(IG)=GL(IE)
        YCC(IG)=YINV(IE)
        D4(IG)=D1(IE)
        WRITE(5,339)ICNODE(IG),ICPODE(IG),IACODE(IG)
```

```
339     FORMAT(1H ,'CC PIPE ',I3,' D/S BN ',I3,' & PIPE ',I2)
        IE=IE-1
        GOTO 300
C
C   PROCEDURE TO REORDER THE BRIDGING PIPE DATA
18      NV=IE+1
        IBCODE(NV)=IBCODE(IE)
        IPCODE(NV)=IPCODE(IE)
        NANI(NV)=NANI(IE)
        XL(NV)=FLOAT(L)
        GL(NV)=GL(IE)
        YINV(NV)=YINV(IE)
        D1(NV)=D1(IE)
        D2(NV)=D2(IE)
        CA(NV)=CA(IE)
        PI(NV)=PI(IE)
        EM(NV)=EM(IE)
        HLI(NV)=HLI(IE)
        XKS(NV)=XKS(IE)
        AIP(NV)=AIP(IE)
        PRP(NV)=PRP(IE)
        FAP(NV)=FAP(IE)
        SI(NV)=SI(IE)
        PAPG(NV)=PAPG(IE)
        ASP(NV)=ASP(IE)
C
        IBCODE(IE)=ICNODE(IG)
        IPCODE(IE)=IRCODE(IG)
        NANI(IE)=NINI(IG)
        XL(IE)=XP(IG)
        GL(IE)=GCC(IG)
        YINV(IE)=YCC(IG)
        D1(IE)=D4(IG)
        IF(XKS(IE).EQ.0)XKS(IE)=RH
C
        IE=NV
        GOTO 6
C
C   OUTFALLS DATA READ-IN
305     IX=IX+1
        IF(IBCODE(IE).GT.-2)GOTO 310
C       READ(I3,1000)I
        READ(I3,159)IBCODE(IE),IPCODE(IE),NANI(IE),YINV(IE)
        ILAST(IX)=IBCODE(IE)
        ILAPE(IX)=IPCODE(IE)
        NENI(IX)=NANI(IE)
        YINT(IX)=YINV(IE)
        GOTO 300
310     IE=IE-1
        READ(I3,159)IBCODE(IE+1),IPCODE(IE+1),NANI(IE+1),YINV(IE+1)
        ILAST(IX)=IBCODE(IE+1)
        ILAPE(IX)=IPCODE(IE+1)
        NENI(IX)=NANI(IE+1)
        YINT(IX)=YINV(IE+1)
        IF(NS.EQ.0) GOTO15
        WRITE(I2,210)NS
        WRITE(5,210)NS
C
```

```
C WRITE OUT INPUT OF DATA FOR IMMEDIATE DATA CHECKING ON VDU
C
15      CONTINUE
        WRITE(5,303)IE
303     FORMAT(1H ,'  THE NO IE (SHOULD = N) IS ',I3,/)
        DO 90 I=1,IE
90      WRITE(5,331)IBCODE(I),IPCODE(I),NANI(I),XL(I)
331     FORMAT(1H ,'BN(DATIN) ',I3,' PIPE ',I3,' NANI ',I2,' L',F6.1)
        DO 91 I=1,IG
        WRITE(5,332)ICCODE(I),IRCODE(I),IWT(I)
332     FORMAT(1H ,'CC BN NO ',I3,' CONN ',I2,' CONN TYPE ',I2)
        WRITE(5,333)ICNODE(I),ICPODE(I),IACODE(I)
333     FORMAT(1H ,'CO BN NO ',I3,' D/S ',I3' PIPE NO ',I3)
91      CONTINUE
        DO 92 I=1,IX
92      WRITE(5,334)ILAST(I),ILAPE(I),NENI(I),YINT(I)
334     FORMAT(/,1X,'O/F ',I3,' PIPE ',I3,' INDEX ',I2,' LEVEL ',F6.3)
C
C   END
```

APPENDIX A(viii)
LEVEL COMPUTATION PROCEDURE

[SUBROUTINE CONST - A(viii)1 & A(viii)2]
[SUBROUTINE FLOW - A(viii)3]



Appendix [A(viii)1]

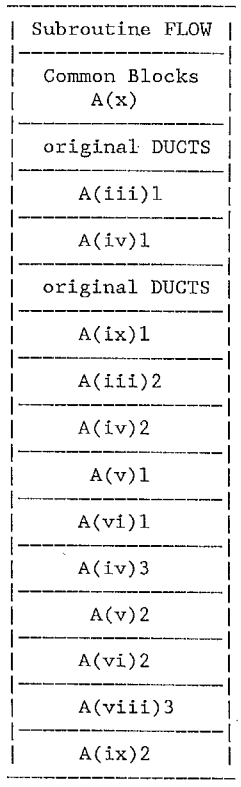
```
C   LEVEL COMPUTATION
C   CALCULATE FULL BORE VELOCITY AND FULL BORE FLOW (QFB)
        D1(I)=IFIX(D)
        SL(I)=S
5       X=XL(I)
        S=SL(I)
        MD=MDOWN(I)
        RK=XKS(I)
        D5=FLOAT(D1(I))/1000.0
C
        IF(IBCODE(I).EQ.700)GOTO 9
        IF(PI(I).NE.0) GOTO 70
C SET UP PARAMETERS FOR CIRCULAR PIPE
C FLOW AND SURFACE WIDTH CALCULATED FOR PROP. DEPTH = 0.6
        D6=D5
        HR=D5*.25
        SW=0.9798*D5
        BSP(I)=0.785398*D5*D5
        GOTO 80
C SET UP PARAMETERS FOR DUNFERMLINE EGG SHAPED PIPES
C PROPORTIONAL DEPTH=0.658
70      CONTINUE
        IF(PI(I).NE.1) GOTO 75
        SW=D5
        D6=(1.5*D5)*0.773
        BSP(I)=1.148532*D5*D5
        WP=3.964947*D5
        HR=BSP(I)/WP
75      CONTINUE
C CALCULATE FULL BORE VELOCITY AND FLOW
C   & THETA (D/KS) FOR EACH PIPE (O. AU-YEUNG ON 20/5/87)
80      CONTINUE
```

```
        F1=SQRT(78.48*HR*S)
        F2=-ALOG10((RK/14800.0+1.255*1.14E-6/F1)/HR)
        VF=2.0*F1*F2
        QF=BSP(I)*VF
C
        THETA=D6/(RK/1000.0)
        IF(THETA.LT.300.0)NDKS(I)=1
        IF(THETA.GE.300.0.AND.THETA.LT.750.0)NDKS(I)=2
        IF(THETA.GE.750.0)NDKS(I)=3
C
```

Appendix [A(viii)2]

```
C
C   PROP DEPTH & DISCHARGE COMPUTATION USING COLEBROOK-WHITE
C       SOLUTIONS FOR PARTIALLY FULL PIPES
C            BY O.W.K. AU-YEUNG ON 20/5/87
C
        PIE=3.141592654
        NPLUS=1
        DO 800 NTA=1,3
        DKS=(FLOAT(NTA*NTA)+1.0)*100.0
        IF(NTA.EQ.1)DKS=100.0
        DO 850 I=1,10000
        DP=FLOAT(I)/10000.0
        TRIG=ACOS(2*DP-1)
        PYE=2*(PIE-TRIG)
        AP=(PYE-SIN(PYE))/(2*PIE)
        ROP=1-(SIN(PYE)/PYE)
C
        QPP=(1+((ALOG10(ROP))/(ALOG10(3.7*DKS))))*AP*(SQRT(ROP))
        QPROP=INT(QPP*1000.0)
        IF(QPROP.LT.1)GOTO 610
        IF(QPROP.GT.1000)GOTO 633
        IF(QPROP.EQ.NPLUS)DPROP(NTA,QPROP)=DP
610     CONTINUE
        NPLUS=QPROP+1
850     CONTINUE
633     CONTINUE
C
        WRITE(I9,418)DKS
418     FORMAT(///,1X,'  THETA (D/KS) = ',F6.1,/)
        WRITE(I9,419)(DPROP(NTA,QPROP),QPROP=1,1000)
419     FORMAT(10F8.4)
800     CONTINUE
C
        DO 880 I=1,N
        WRITE(I9,424)D1(I),NDKS(I)
424     FORMAT(1H ,10X,'DIA = ',I4,'  NTHE = ',I2)
880     CONTINUE
C
C ***************************************************************************
C
        END
C
C ***************************************************************************
```

[SUBROUTINE FLOW]



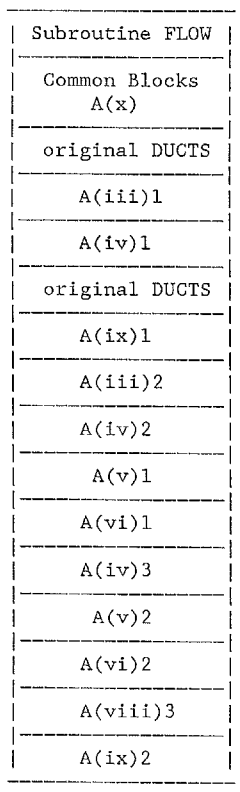
Appendix [A(viii)3]

```
C    FLOW ROUTING FOR FREE SURFACE FLOW
C    MUSKINGUM ROUTING
          Q2=C1(J)*QUP(J)+C2(J)*QW(J)+C3(J)*QDN(J)
33        IF(ABS(Q2).LT.0.0000001) Q2=0.0
          IF(Q2.LT.0.0)Q2=0.0
C         IND=INT(Q2/QFB(J))*1000
          RAQ=(Q2/QFB(J))*1000.0
          IND=IFIX(RAQ)
          IF(IND.LE.0)YNN=0.0
          IF(IND.LE.0)GOTO 299
          IF(IND.EQ.1000)YNN=FLOAT(D1(J))/1000.0
          IF(IND.EQ.1000)GOTO 299
          IF(IND.GT.1000)GOTO 50
```

```
      YNN=PDIA*DPROP(NDKS(J),IND)
c     WRITE(IBELOW,124)IT,IH,J,QDN(J),QUP(J),QW(MD)
      GOTO 299
C
C
C   LEVEL COMPUTATION PROCEDURE ENDS
```

APPENDIX A(ix)
PRESSURISED FLOW ROUTING FOR PARALLEL PIPES

[SUBROUTINE FLOW]



Appendix [A(ix)1]

```
C  TO CHECK SURCHARGE IN PIPE BY WATER LEVEL
C    INEQUALITY 17
       HTEST=BETA(J)*Q*Q-(Y(J)-Y(MD))
       YY=0.0
       IF(HTEST.LE.-.001) GOTO20
C HTEST > 0 == PIPE SURCHARGED
       YY=Y(MD)+BETA(J)*Q*Q
C  -- FREE SURFACE FLOW ROUTING PROCEDURE
       IF (YY.GT.YINV(J)) GOTO 50
```

Appendix [A(ix)2]

```
C ------------------------------------------------
C  SURCHARGE MODEL FOR DUNFERMLINE PARALLEL PIPE
C       MODIFIED BY OLLY AU-YEUNG
C       FIRST ON    15-7-87, Dec 1988
C       Further in Jan, Feb 1989
C ------------------------------------------------
C A SURCHARGED PIPE HAS BEEN IDENTIFIED
50      CONTINUE
        ISW=1
C ISW = 0 IF NO SURCHARGING EVER OCCURS
        ISTACK=ISTACK+1
        NNN=N+1
        if(istack.GT.MEG) call error(I1,505)
        IF(ISTACK.EQ.1) GOTO 65
        DO 60 I=1,ISTACK-1
        II=ISTACK-I
60      NSTACK(II+1)=NSTACK(II)
65      NSTACK(1)=J
C
C       ADD-ON ON 23/7/87
        IF(NANI(MD).EQ.6)GOTO 68
        IF(IBCODE(J+1).EQ.IDBODE(KP).AND.IPCODE(J+1).EQ.IDPODE(KP))
     1 GOTO 83
        GOTO 82
83      KP=KP+1
        GOTO 68
82      CONTINUE
C TEST DOWNSTREAM NODE TO CHECK IF IT IS SURCHARGED
        YT=Y(MD)-YINV(MD)
        IF(QUP(MD).GT.QFB(MD).OR.YT.GT.0.005) GOTO 300
C       A SURCHARGED SUBSYSTEM HAS NOW BEEN IDENTIFIED
68      CONTINUE
        DO 681 I=1,N
        QZ(I)=0.0
681     YS(I)=0.0
C       SET UP SUBSYSTEM MATRICES
        IF(IH.NE.1)GOTO 67
        ITTACK=ITTACK+ISTACK
        IUTACK=IUTACK+1
        WRITE(5,1075)ISTACK
        MEMT(1,1)=1
        IF(ISTACK.EQ.1)GOTO90
C SET SURCHARGED SUBSYSTEM MATRIX TO ZERO
67      CONTINUE
        DO 70 II1=1,ISTACK
        DO 70 JJ1=1,ISTACK
        MEMT(II1,JJ1)=0
70      CONTINUE
C THE FOLLOWING PROCEDURE WAS SET UP ON 29TH AUGUST 1986
C TO SIMPLIFY AND REDUCE STORAGE FOR SURCHARGED SUBSYSTEM
C BBC PROGRAM MATRIX IS FULLY DOCUMENTED
        DO 90 II1=1,ISTACK
        MEMT(II1,II1)=1
        MP=NSTACK(ISTACK+1-II1)
        MD=MDOWN(MP)
```

```
        DO 85 KK=1,ISTACK
        MK=NSTACK(ISTACK+1-KK)
        IF(MK.NE.MD) GOTO 85
        MEMT(KK,II1)=-1
85      CONTINUE
90      CONTINUE
C SURCHARGED SUBSYSTEM STRUCTURE MATRICES HAVE NOW BEEN SET UP
131     CONTINUE
C STEADY STATE SOLUTION TO SURCHARGED SUBSYSTEM
C
        QZ(1)=QW(NSTACK(ISTACK))
        IF(ISTACK.EQ.1) GOTO 137
        DO 135 JJ=2,ISTACK
        T=0.0
        DO 134 J1=1,JJ-1
134     T=T+FLOAT(MEMT(JJ,J1))*QZ(J1)
135     QZ(JJ)=QW(NSTACK(ISTACK+1-JJ))-T
C QZ( ) CONTAINS STEADY STATE DISCHARGES
C
C NOW COMPUTE STEADY STATE LEVELS IN SURCHARGED SUBSYSTEM
C
137     CONTINUE
138     MD=MDOWN(NSTACK(1))
        YS(MD)=Y(MD)
        DO 140 JJ=1,ISTACK
        J1=NSTACK(JJ)
        MD=MDOWN(J1)
        ZZ=QZ(ISTACK+1-JJ)
        YS(J1)=ZZ*ZZ*BETA(J1)+YS(MD)
140     CONTINUE
C
C YS( ) CONTAINS STEADY STATE LEVELS
C
C START SOLVING THE SURCHARGED SUBSYSTEM
C THE TIMESTEP H IN THE SOLUTION WILL NORMALLY BE 1 SEC
C SINCE THE OVERALL TIMESTEP IS 10 SECS THEN THIS CONTINUES FOR 10 STEPS
C
        H=1.0
        DO 250 IHH=1,10
C*********************************************************************
C
C START OF SOLUTION OF SURCHARGED PIPE SUBSYSTEM USING
C------------STANDARD RUNGE KUTTA-----------------------------
C
C*********************************************************************
        DO 200 II=1,ISTACK
C CHECK AGAINST STEADY STATE SOLUTION
        NI=NSTACK(ISTACK+1-II)
        RKCHK=ABS(YS(NI)-Y(NI))
        IF(RKCHK.LE.RKACC) GOTO 200
C FIND OUT LOCATIONS IN MEMT WITH NON ZEROS
        ICN=0
        IF(ISTACK.LE.3) GOTO 1450
1450    CONTINUE
        DO 150 LI=1,ISTACK
        MM=MEMT(II,LI)
        IF(MM.EQ.0) GOTO 150
        ICN=ICN+1
```

```
        MT(ICN)=MM
        NT(ICN)=LI
150     CONTINUE
C
C NOW PROCEED WITH RUNGE KUTTA SOLUTION
C
        Q1=QW(NI)
        ALPH=ALPHA(NI)
        YKA=0.0
        YK1=0.0
        IT22=IT
        CALL ZSOL(Q1,ALPH,H,YKA,YK1,ICN,IT22)
        YKA=YK1/2.0
        CALL ZSOL(Q1,ALPH,H,YKA,YK2,ICN,IT22)
        YKA=YK2/2.0
        CALL ZSOL(Q1,ALPH,H,YKA,YK3,ICN,IT22)
        YKA=YK3
        CALL ZSOL(Q1,ALPH,H,YKA,YK4,ICN,IT22)
        DY=(YK1+2.0*YK2+2.0*YK3+YK4)/6.0
        Y(NI)=Y(NI)+DY
C calculate any surface flooding
        IF(Y(NI).LE.GL(NI)) GOTO 190
        Y(NI)=GL(NI)
        ROREA(NI)=ROREA(NI)+DY*ALPH
        AIMP(NI)=1
        NSURF=1
190     CONTINUE
C water level cannot go above ground level
        IF(Y(NI).GT.GL(NI))Y(NI)=GL(NI)
C water level cannot go below invert
        IF(Y(NI).LT.YINV(NI)) Y(NI)=YINV(NI)
C Is current level the highest yet?
        IF(Y(NI).GT.XKS(NI)) XKS(NI)=Y(NI)
D       IF(IT.NE.33) GOTO 181
c       WRITE(ISURCH,170)YINV(NI),Y(NI),YS(NI),DY,ISTACK,NSTACK(ISTACK),
c      1nSTACK(1)
D181    CONTINUE
C
C END OF SURCHARGED SUBSYSTEM MODEL
200     CONTINUE
C
C END OF SUBTIMELOOP FOR SURCHARGED SUBSYSTEM
250     CONTINUE
        IF(ISW.EQ.0)GOTO 280
c       WRITE(ISURCH,171) IT,IH,ISTACK
c       WRITE(ISURCH,171) NSTACK(ISTACK),NSTACK(1)
280     CONTINUE
C
C END OF PIPE LOOP FOR COMPLETE SYSTEM
C
C STORE FLOWS FROM SURCHARGED SUBSYSTEM IN ARRAY QUP( )
        DO 270 IK=1,ISTACK
        IK1=NSTACK(IK)
        MD=MDOWN(IK1)
C   WATCH THE FOLLOWING STATEMENT --- PUT IN ON 24/11/87 (OLLY)
        YN(IK1)=Y(IK1)-YINV(IK1)
        IF(YN(IK1).LT.0.0000001)YN(IK1)=0.0
```

```
        YW(IK1)=Y(IK1)
C   END OF TEMP INPUT
        IF(IBCODE(IK1).NE.700)GOTO 620
C
C ----------------------------------------
C      CROSS CONNECTION OVERFLOW  (VR. 3)
C       pressurised flow condition
C ----------------------------------------
        NH=NH+1
        MDD=MORY(NH)
        MDE=MDD-1
        YLV=YN(J)
        NNF=MORE(NH)
        QFBF=QFB(NNF)
        QFBS=QFB(MDD)
        QFBB=QFB(MDE)
        YSTR=YSTO(MDD)
        TD1=FLOAT(D1(NNF))/1000.0
        STDIA=STSL(NH)-STIL(NH)
        CRDIA=CRSL(NH)-CRIL(NH)
        IDD1=D4(NH)
        TD2=(FLOAT(IDD1))/1000.0
        CNDIA=CNSL(NH)-YCC(NH)
        ID2=D3(NH)
        DD1=(FLOAT(ID2))/1000.0
        CSAF=ARAF(NH)
        CSAS=ARAS(NH)
        CSAB=ARAN(NH)
        YCOMP=DD1-TD1
C
        QFIN=QW(J)
        QFST=QFIN
        QSTR=QCOLD(J)
        QCRO=QOVE(J)
        VCOLD=VCC(J)
C
        YCO=YW(IK1)
        IF(YCO.GT.DD1)GOTO 628
        QCRO=0.0
        GOTO 625
C
628     CONTINUE
        YRLF=YCO+STIL(NH)
        YRLS=YSTR+CRIL(NH)
        IF(YRLS.GT.YRLF)GOTO 681
        IF(YRLS.EQ.YRLF)GOTO 685
        IF(YRLS.GE.BRLOT)GOTO 623
C
630     CONTINUE
        YHW=YCO-DD1
        YHAV=0.083*(YHW/DD1)
        CD1=0.602+YHAV
        IF(IWT(NH).NE.0)CD1=1.204+YHAV
        YCHD=YHW**1.5
        QCRO=CD1*WL(NH)*3.13*YCHD
        IF(QCRO.LE.QFBB)GOTO 683
        YBEN=YCO-(0.5*TD2)
        QCRO=0.6*CSAB*SQRT(19.62*YBEN)
```

```
                                                                      A(ix)
683     CONTINUE
        YCEN=YCO-(0.5*TD1)
        IF(YCEN.LE.0.0)YCEN=0.0
        QSTR=0.6*CSAF*SQRT(19.62*YCEN)
686     CONTINUE
        QCOUT=QSTR+QCRO
        IF(ABS(QFIN-QCOUT).LE.0.001)GOTO 690
        IF(QCOUT.GT.QFIN)YCO=0.99*YCO
        IF(QCOUT.LT.QFIN)YCO=1.01*YCO
        GOTO 630
C
623     YDIFF=YRLF-YRLS
        QCRO=YDIFF*SQRT(ABS(YDIFF)/BETA(IK1))/ABS(YDIFF)
        QSTR=QFIN-QCRO
690     CONTINUE
        YCO=QSTR/AMHF
        QREV=0.0
        GOTO 625
C
685     CONTINUE
        QCRO=0.0
        QREV=0.0
        GOTO 625
C
681     CONTINUE
        YDIFF=YRLS-YRLF
        QREV=YDIFF*SQRT(ABS(YDIFF)/BETA(IK1))/ABS(YDIFF)
        QSTR=QFIN+QREV
        YCO=QSTR/AMHF
        QCRO=0.0
C
625     CONTINUE
        QCRS(NH)=QCRO
        QOVE(J)=QCRO
        YNN=YCO
        QCRS(NH)=-1.0*QREV
620     CONTINUE
        YYY=Y(IK1)-Y(MD)
        QUP(IK1)=YYY*SQRT(ABS(YYY)/BETA(IK1))/ABS(YYY)
C       QUP(IK1)=ABS(YYY)*SQRT(ABS(YYY)/BETA(IK1))/ABS(YYY)
        QDN(IK1)=QUP(IK1)
        QW(IK1)=QUP(IK1)
270     CONTINUE
C
C  SURCHARGE FLOW ROUTING PROCEDURES FOR PARALLEL PIPES END HERE
```

APPENDIX A(x)
COMMON BLOCKS

AT THE BEGINNING OF EACH SUBROUTINE

```
C   MAXIMUM NUMBER OF SYSTEM DATA LINES IS 450
C    PERMITTED NUMBER OF CROSS-CONNECTIONS IS 10 CURRENTLY
C    MAXIMUM RAINFALL TIME IS 12 HOURS
C    MAXIMUM FLOW ROUTING TIME-STEPS IS 1556 MINUTES
        PARAMETER MEN=450
        PARAMETER MEG=100
        parameter MEJ=730
        parameter mek=mej+100
        parameter meh=mek*3/2
        parameter mei=meh*5/4
        CHARACTER*1 NANS
        CHARACTER RTITL*30,RDATE*10,RTIME*10,SC*10
        CHARACTER*12 ifile,rfile
        INTEGER D1,D2,PAPG,AIP,PRP,FAP,PI,EM,HLI,SI,HYD,D3,D4,QPROP
        INTEGER STORM
        COMMON/SYSDAT/N,NA,DT,TITLE(16),in,mnords,nst,RP,ISTACK,D7(2),MC
        COMMON/device/I1,I2,I3,I4
      1,ibelow,meni,megi,mehi,meji,MEKI,NV,IX,NB,IG,NQ
        COMMON/FILENAMES/IFILE,RFILE,IGRF,ISURF,ISUBA,ISURCH
        COMMON/ORDER/NENDO(MEN),IBCODE(MEN),IPCODE(MEN),MDOWN(MEN)
      1,ICCODE(20),IRCODE(20),ICNODE(20),ICPODE(20),IACODE(20)
      2,LDOWN(20),ILAST(5),ILAPE(5),IAN(20),GCO(20),IWT(20)
      3,MDNEW(MEN),MORE(20),MORY(20),IUB(20),IUP(20)
        COMMON/PIPE1/YINV(MEN),D1(MEN),D2(MEN),PI(MEN),EM(MEN),HLI(MEN)
      1,D3(20),D4(20),YINC(20),YINT(5),WL(20),SLB(20),D8(MEN)
        COMMON/PIPE2/SL(MEN),BETA(MEN),XL(MEN),XKS(MEN),NDKS(MEN)
        COMMON/SURF/GL(MEN),CA(MEN),AIP(MEN),PRP(MEN),FAP(MEN),SI(MEN)
      1,PAPG(MEN),CD(20),IQSET(20)
        COMMON/GEN/NANI(MEN),DWF(MEN),DRY(MEN),ALPHA(MEN),NSTEX
      1,NINI(20),NENI(5)
        COMMON/CONNECT/STIL(20),CRIL(20),STSL(20),CRSL(20),CNSL(20)
      1,WLEV(20),ARAF(20),ARAS(20),ARAN(20),BNSL(20)
        COMMON/OVER/IOV(MEN),QOV(MEN),JOV(10),KOV(10),LOV(10),OVA(10)
      1,OVB(10),ovc(10,mei),IOVF,IO,NSSA,GCC(20),YCC(20),XP(20)
        COMMON/OFFLT/IOFF(MEN),QOFF(MEN),JOFF(5),KOFF(5),LOFF(5),OFFA(5)
      1,OFFB(5),QCOLD(MEN),VCC(MEN),QOVE(MEN)
        COMMON/ONLT/ION(MEN),QON(MEN),JON(5),KON(5),LON(5),ONA(5),ONB(5)
      1,IST(5),ISCODE(5),ITCODE(5),IDBODE(5),IDPODE(5),TBL(5),TD(5),TL(5)
      2,TCOF(5),PAREA(5),TW1(5),TW2(5),TOD(5),TCD(5),OLT(5,MEI),IOL
      3,VOLT(MEN),V1(MEN),Y1(MEN),QOLD(MEN)
        COMMON/SPARE/ASP(MEN),BSP(MEN),CSP(MEN),ISP(MEN),dsp(10,mei)
      1,esp(10,mei),HLEL(20,MEI)
        COMMON/EXT/QFB(MEN),QOUT(20,mei),IOUT(10),IOUTJ,QCRS(MEN)
        COMMON/MUSK/QW(MEN),QUP(MEN),QDN(MEN),C1(MEN),C2(MEN),C3(MEN)
        COMMON/END/IJM(10),IJS(10),NJC1(10),NSTACK(MEN),LSTACK(10)
        COMMON/SUBSYS/MEMT(MEG,MEG),DPROP(3,1000),YN(MEN),YUP(MEN),YW(MEN)
        COMMON/RUNGE/Y(MEN),AINF(MEN),YS(MEN),NT(50),MT(50),QZ(MEN)
        COMMON/HYDRO/hydro(10,meh),PRROOF,SATIS,RF,STORM,PRPAV,PRPRV
      1,API5,SMD,PIMP,SOIL,SAAR,AT,ucwi,pr,spr,NTSR,NEVT
        COMMON/MISC/RTITL,RTIME,RDATE
        COMMON/AREA/EARROF(MEN),EARPRV(MEN),EARPVD(MEN),ROREA(MEN)
      1,AIMP(MEN),PEREA(MEN),HYD(MEN)
```

```
                                                                      A(x)

      COMMON/PIP/INFLOW,INC,SUB(50,MEH),DIA(40),CS1(40),CS2(40),
     1CS3(40),AK1(40),AK2(40),AK3(40),QP(10),QD(10)
      COMMON/RAINF/RAIN(mej),RR(meh),RAIN1(mej),REC1(mej)
CCCCC DOUBLE PRECISION IFILE,RFILE
      DATA I1,I2,I3,I4,IGRF/24,23,21,22,20/
      data meni,megi,mehi,meii,meji,meki/men,meg,meh,mei,mej,mek/
C
C ***************************************************************
C
```

APPENDIX A(xi)

SAMPLE OF SSD DATA INPUT FILE

```
BOTHWELL ST CATCHMENT TOTAL INFLOW -- PARALLEL PIPE VERSION
 6.000000
 0.000000
780
 0.450000
  1
 10
  5
  2
 1.000000
 1   0 2  325   0.00  0.0243  300     0 6.520 0 0 0   0.0 40  0  5 2-2  4.000
 1   1 0  130  97.98 94.2320  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 1   2 0   25  96.88 93.8320  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 2   0 2  311   0.00  0.0435  225     0 3.780 0 0 0   0.0 40  0  5 0 0  0.300
 1   3 0   25  96.40 93.6080  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 1   4 0   20  95.84 93.5580  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 3   0 0  151 105.11103.3820  225   225 0.000 0 0 0   0.0  0  0  0 3 0  0.200
 1   5 0   90  94.89 93.3180  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 1   6 0   57  94.59 93.1410  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 1   7 0   10  95.11 92.9240  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 4   0 0  273 106.52104.2520  225   225 0.000 0 0 0   0.0  0  0  0 0 0  0.200
 1   8 0   82  95.29 92.8510  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 1   9 0  217  96.51 92.5710  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 5   0 2  497   0.00  0.0182  375     0 4.430 0 0 0   0.0 27  0  5 1 0  0.800
 6   0 2  534   0.00  0.0046  300     0 2.310 0 0 0   0.0 14 11  5 1 0  0.600
 6   1 0   20  84.38 83.5430  375   375 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 1  10 0   93  84.32 82.7950  375   375 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 7   0 2 1010   0.00  0.0340  375     029.088 0 0 0   0.0 27 13  5 3 0  2.000
 7   1 0  230 100.12 98.6880  375   375 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 7   2 0   75  98.60 94.9450  375   375 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 7   3 0   90  97.12 93.5740  300   300 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 7   4 0   90  95.29 91.5860  300   300 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 7   5 0   20  92.33 88.3190  300   300 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 8   0 2   52   0.00  0.0220  300     0 2.060 0 0 0   0.0  2 21  5 1 0  0.600
 9   0 2   47   0.00  0.0292  300     0 0.490 0 0 0   0.0 31  0  5 1 0  0.200
 7   6 0   85  88.41 85.2590  375   375 0.430 0 0 0   0.0  9  0  2 1 0  0.200
10   0 2   52   0.00  0.0343  225     0 0.310 0 0 0   0.0 11  0  2 1 0  0.200
 7   7 0    9  86.18 85.0010  375   375 0.000 0 0 0   0.0  0  0  0 0 0  0.000
11   0 2  292   0.00  0.0878  150     0 3.730 0 0 0   0.0 28  0  2 0 0  0.800
 7   8 0   63  86.51 84.7430  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
12   0 2  230   0.00  0.0824  150     0 2.160 0 0 0   0.0 28  0  0 0 0  0.600
 7   9 0   75  86.46 84.1580  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
13   0 2   90   0.00  0.0183  150     0 0.590 0 0 0   0.0 32  0  2 1 0  0.100
14   0 2  215   0.00  0.0469  150     0 1.470 0 0 0   0.0 26  0  0 0 0  0.200
 7  10 0   47  85.89 83.4660  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
15   0 2   94   0.00  0.0165  200     0 0.690 0 0 0   0.0 30  0  2 1 0  0.100
 7  11 0   84  85.18 83.2040  450   450 0.490 0 0 0   0.0  3 14  2 1 0  0.100
16   0 2  100   0.00  0.0160  150     0 0.880 0 0 0   0.0  3 10  0 1 0  0.100
 7  12 0   78  84.94 82.5090  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
17   0 2  310   0.00  0.0383  225     0 3.830 0 0 0   0.0 20 10  0 0 0  0.800
 7  13 0   22  84.42 82.3390  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 7  14 0   87  83.59 81.7440  450   450 1.080 0 0 0   0.0 58  0  2 1 0  0.200
18   0 2  175   0.00  0.0470  225     0 3.730 0 0 0   0.0 58  0  2 1 0  0.800
 7  15 0   25  84.14 81.0890  600   600 0.000 1 0 0   0.0  0  0  0 0 0  0.000
 7  16 0   31  83.64 80.7420  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 7  17 0   36  83.23 80.6264  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 7  18 0   45  83.06 80.5108  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 7  19 0   83  83.18 80.3952  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 7  20 0   77  82.96 80.2796  450   450 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 1  11 0   78  83.34 80.1640  600   600 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 1  12 0   46  82.11 78.1380  600   600 0.000 0 0 0   0.0  0  0  0 0 0  0.000
19   0 0   20  84.86 79.6210  150   150 0.000 0 0 0   0.0  0  0  0 0 0  0.000
19   1 0   38  84.33 79.1824  150   150 0.000 0 0 0   0.0  0  0  0 0 0  0.000
19   2 0    7  83.75 78.7438  150   150 0.000 0 0 0   0.0  0  0  0 0 0  0.100
19   3 0   32  83.11 78.3051  225   225 0.000 0 0 0   0.0  0  0  0 0 0  0.100
19   4 0   48  82.68 77.8665  225   225 0.000 0 0 0   0.0  0  0  0 0 0  0.100
19   5 0   14  82.54 77.4279  225   225 0.000 0 0 0   0.0  0  0  0 0 0  0.100
19   6 0   32  82.02 76.9893  225   225 0.000 0 0 0   0.0  0  0  0 0 0  0.100
19   7 0   23  81.74 76.5506  225   225 0.000 0 0 0   0.0  0  0  0 0 0  0.100
 1  13 0   60  81.53 76.1120  600   600 0.000 0 0 0   0.0  0  0  0 0 0  0.000
 1  14 0   98  77.98 74.3000  600   600 0.000 0 0 0   0.0  0  0  0 0 0  0.000
20   0 0   58  80.63 77.3230  225   225 0.000 0 0 0   0.0  0  0  0 0 0  0.200
20   1 0   66  77.54 74.9055  225   225 0.000 0 0 0   0.0  0  0  0 0 0  0.200
 1  15 0  125  74.44 72.4880  600   600 0.000 0 0 0   0.0  0  0  0 0 0  0.000
21   0 0   35  84.98 83.2230  150   150 0.000 0 0 0   0.0  0  0  0 0 0  0.000
21   1 0   51  83.93 82.7700  150   150 0.000 0 0 0   0.0  0  0  0 0 0  0.100
21   2 0    7  82.87 82.3170  150   150 0.000 0 0 0   0.0  0  0  0 0 0  0.100
21   3 0   55  81.82 81.0595  150   150 0.000 0 0 0   0.0  0  0  0 0 0  0.100
21   4 0   38  80.76 79.8020  225   225 0.000 0 0 0   0.0  0  0  0 0 0  0.100
```

																						A(xi)
21	5	0	10	79.71	79.2230	225	225	0.000	0	0	0	0.0	0	0	0	0	0					0.100
21	6	0	43	78.65	79.0400	225	225	0.000	0	0	0	0.0	0	0	0	0	0					0.100
21	7	0	59	77.60	77.4460	300	300	0.000	0	0	0	0.0	0	0	0	0	0					0.100
21	8	0	53	76.54	74.3680	300	300	0.000	0	0	0	0.0	0	0	0	0	0					0.100
1	16	0	66	73.68	71.3290	600	600	0.000	0	0	0	0.0	0	0	0	0	0					0.000
22	0	0	100	104.89	103.2510	150	150	0.000	0	0	0	0.0	0	0	0	0	0					0.200
22	1	0	100	96.92	94.9840	150	150	0.000	0	0	0	0.0	0	0	0	0	0					0.200
22	2	0	100	88.95	86.7170	150	150	0.000	0	0	0	0.0	0	0	0	0	0					0.300
22	3	0	100	80.97	78.4500	150	150	0.000	0	0	0	0.0	0	0	0	0	0					0.300
1	17	0	96	73.00	70.1830	600	600	0.000	0	0	0	0.0	0	0	0	0	0					0.000
1	18	0	102	72.24	69.5680	600	600	0.000	0	0	0	0.0	0	0	0	0	0					0.000
1	19	0	188	71.70	68.7570	600	600	0.000	0	0	0	0.0	0	0	0	0	0					0.000
23	0	0	671	84.58	82.9820	450	450	0.000	0	5	0	0.0	0	0	0	0	0					0.200
24	0	2	667	0.00	0.0308	300	0	9.540	0	0	0	0.0	58	0	5	3	0					4.000
1	20	0	124	70.27	67.6470	600	600	0.000	0	0	0	0.0	0	0	0	0	0					0.000
1	21	0	117	69.20	66.2820	600	600	0.000	0	0	0	0.0	0	0	0	0	0					0.000
25	0	0	450	89.50	87.9100	375	375	0.000	0	3	0	0.0	0	0	0	0	0					0.800
1	22	0	52	68.33	64.7460	600	600	0.000	0	0	0	0.0	0	0	0	0	0					0.000
1	23	3	94	69.86	64.1020	600	600	0.000	1	0	0	0.0	0	0	0	0	0					0.000
700	1	3	1	69.37	60.7890	450	0	0.000	0													
800	600	0	3	59.79	60.6430	700	700	0.000	0	0	0	0.0	0	0	0	0	0					0.000
500	0	0	96	69.37	60.6430	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000
26	0	0	601	107.27	106.2990	450	450	0.000	0	4	0	0.0	0	0	0	0	0					0.400
26	1	0	72	77.27	74.8290	450	450	0.000	0	1	0	0.0	0	0	0	0	0					0.000
27	0	0	657	109.15	105.7660	450	450	0.000	0	4	0	0.0	0	0	0	0	0					0.400
26	2	0	277	74.31	71.4600	450	450	0.000	0	4	0	0.0	0	0	0	0	0					0.000
28	0	0	168	76.88	75.1420	375	375	0.000	0	3	0	0.0	0	0	0	0	0					0.400
26	3	0	80	71.70	69.7360	475	475	0.000	0	2	0	0.0	0	0	0	0	0					0.000
500	1	0	47	67.59	59.1590	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000
500	2	0	104	68.41	58.6710	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000
29	0	2	675	0.00	0.0510	375	0	12.168	0	0	0	0.0	58	0	5	3	0					1.200
500	3	0	87	65.05	57.7260	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000
30	0	2	215	0.00	0.0751	225	0	1.290	0	0	0	0.0	24	0	0	3	0					0.200
500	4	0	105	64.32	57.1130	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000
31	0	2	426	0.00	0.0862	300	0	9.369	0	0	0	0.0	58	0	5	3	0					3.100
500	5	0	160	63.13	56.3610	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000
600	0	0	94	69.37	59.7930	800	800	0.000	0	0	0	0.0	0	0	0	0	0					0.010
600	1	0	83	67.61	58.4730	850	850	0.000	0	0	0	0.0	0	0	0	0	0					0.010
600	2	0	66	68.11	57.6380	850	850	0.000	0	0	0	0.0	0	0	0	0	0					0.000
600	3	0	88	64.95	56.8880	900	900	0.000	0	0	0	0.0	0	0	0	0	0					0.000
600	4	0	108	64.42	56.1230	900	900	0.000	0	0	0	0.0	0	0	0	0	0					0.000
600	5	0	149	62.98	54.9310	975	975	0.000	0	0	0	0.0	0	0	0	0	0					0.000
32	0	2	314	0.00	0.0544	300	0	3.760	0	0	0	0.0	24	0	3	3	0					0.800
700	2	0	1	61.34	55.7700	350	0	0.000	0													
800	600	6	2	55.35	55.5900	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000
500	6	0	102	61.34	55.5900	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000
500	7	0	131	59.65	54.6630	375	375	0.000	0	0	0	0.0	0	0	0	0	0					0.000
600	6	0	99	61.62	53.9400	975	975	0.000	0	0	0	0.0	0	0	0	0	0					0.000
600	7	0	80	59.10	53.3520	850	850	0.000	0	0	0	0.0	0	0	0	0	0					0.000
600	8	0	62	56.59	51.3590	825	825	0.000	0	0	0	0.0	0	0	0	0	0					0.000
700	3	0	2	58.76	52.8800	375	0	0.000	1													
800	600	9	4	50.92	51.2150	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000
500	8	0	7	58.76	52.6700	375	375	0.000	0	0	0	0.0	0	0	0	0	0					0.000
33	0	2	246	0.00	0.0875	300	0	2.970	0	0	0	0.0	24	0	3	3	0					0.800
500	9	0	78	59.12	51.9420	375	375	0.000	0	0	0	0.0	0	0	0	0	0					0.000
500	10	0	108	54.22	49.5450	375	375	0.000	0	0	0	0.0	0	0	0	0	0					0.000
500	11	0	98	51.38	48.7590	375	375	0.000	0	0	0	0.0	0	0	0	0	0					0.000
34	0	0	264	63.87	60.5440	225	225	0.000	0	3	0	0.0	0	0	0	0	0					0.800
35	0	0	177	61.75	59.8860	150	150	0.000	0	3	0	0.0	0	0	0	0	0					0.800
36	0	2	627	0.00	0.0468	300	0	6.300	0	0	0	0.0	40	0	5	3	0					6.000
500	12	0	190	50.10	47.9740	375	375	0.000	0	0	0	0.0	0	0	0	0	0					0.000
37	0	2	212	0.00	0.0321	225	0	1.459	0	0	0	0.0	18	0	2	1	0					0.000
500	13	0	56	47.93	46.1770	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000
600	9	0	117	59.01	50.3150	1050	1050	0.000	0	0	0	0.0	0	0	0	0	0					0.000
600	10	0	111	51.74	48.0060	1050	1050	0.000	0	0	0	0.0	0	0	0	0	0					0.000
600	11	0	111	51.51	47.2290	1050	1050	0.000	0	0	0	0.0	0	0	0	0	0					0.000
600	12	0	127	49.96	46.4060	1050	1050	0.000	0	0	0	0.0	0	0	0	0	0					0.000
600	13	0	98	49.88	45.4760	1050	1050	0.000	0	0	0	0.0	0	0	0	0	0					0.000
38	0	0	68	54.88	53.4920	375	375	0.000	0	0	0	0.0	0	0	0	1	0					0.000
39	0	2	206	0.00	0.0560	150	0	2.358	0	0	0	0.0	20	0	2	3	0					1.800
38	1	0	88	55.81	52.6570	375	375	0.000	0	0	0	0.0	0	0	0	1	0					0.000
38	2	0	90	53.73	51.5130	375	375	0.000	0	0	0	0.0	0	0	0	1	0					0.000
40	0	2	276	0.00	0.0444	300	0	3.438	0	0	0	0.0	20	0	2	2	0					1.900
41	0	2	292	0.00	0.0249	225	0	3.242	0	0	0	0.0	20	5	2	1	0					1.900
38	3	0	91	51.21	50.3680	375	375	0.000	0	0	0	0.0	0	0	0	1	0					0.000
38	4	0	88	50.72	49.4170	375	375	0.000	0	0	0	0.0	0	0	0	1	0					0.000
42	0	2	163	0.00	0.0193	150	0	3.045	0	0	0	0.0	15	5	2	1	0					1.800
38	5	0	81	50.23	48.5250	375	375	0.000	0	0	0	0.0	0	0	0	1	0					0.000
38	6	0	117	49.12	47.6740	375	375	0.000	0	0	0	0.0	0	0	0	1	0					0.000
38	7	0	13	48.20	46.4210	300	300	0.000	0	0	0	0.0	0	0	0	1	0					0.000
700	4	0	2	48.22	45.9150	400	0	0.000	0													
800	600	14	3	44.86	45.8750	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000
500	14	0	88	48.22	45.8750	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000
500	15	0	56	47.92	45.0740	375	375	0.000	0	0	0	0.0	0	0	0	0	0					0.000
43	0	2	680	0.00	0.0298	450	0	7.171	0	0	0	0.0	30	10	2	2	0					0.000
43	1	0	93	69.23	67.1170	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000
43	2	0	190	68.62	66.3720	450	450	0.000	0	0	0	0.0	0	0	0	0	0					0.000

A(xi)

```
 44    0 0   246  69.00 67.8100  225   225 0.000 0 4 0  0.0  0  0  0 2 0   0.000
 43    3 0    43  68.96 64.9200  450   450 0.000 0 0 0  0.0  0  0  0 0 0   0.000
 43    4 0    50  67.71 64.5380  450   450 0.000 0 0 0  0.0  0  0  0 0 0   0.000
 43    5 0    80  66.78 63.5360  450   450 0.000 0 0 0  0.0  0  0  0 0 0   0.000
 45    0 2   182   0.00  0.0786  450     0 5.206 0 0 0  0.0 20  8  2 2 0   0.000
 43    6 0    68  65.07 62.1910  450   450 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 46    0 0   843  76.39 74.8100  225   225 0.000 0 5 0  0.0  0  0  0 1 0   0.000
 43    7 0    56  64.19 61.1580  450   450 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 47    0 0   619  73.58 71.3230  150   150 0.000 0 5 0  0.0  0  0  0 2 0   0.000
 48    0 2   569   0.00  0.0464  375    013.851 0 0 0  0.0 30 10  2 3 0   0.000
 43    8 0    23  63.00 60.9930  450   450 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 43    9 0    37  62.68 60.5380  450   450 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 49    0 0   215  69.74 67.9920  150   150 0.000 0 3 0  0.0  0  0  0 2 0   0.000
 43   10 0    53  62.30 60.0210  838   838 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 43   11 0   104  61.89 59.4120  838   838 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 43   12 0    55  61.28 58.5490  838   838 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 50    0 2   585   0.00  0.0597  300    011.198 0 0 0  0.0 30 10  2 3 0   0.000
 51    0 0   509  69.56 68.1990  309   309 0.000 0 5 0  0.0  0  0  0 2 0   0.000
 43   13 0    37  60.76 57.4490  900   900 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 43   14 0   100  58.98 56.6310  900   900 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 52    0 0   444  67.97 66.7150  300   300 0.000 0 6 0  0.0  0  0  0 1 0   0.000
 43   15 0   112  57.64 55.8130  900   900 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 53    0 2   374   0.00  0.0802  375     0 8.644 0 0 0  0.0 30 10  2 3 0   0.000
 54    0 2   206   0.00  0.0461  300     0 3.210 0 0 0  0.0 25  5  2 2 0   0.000
 43   16 0   147  55.87 54.3260  900   900 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 55    0 2   499   0.00  0.0669  450     0 5.599 0 0 0  0.0 25 10  2 3 0   0.000
 43   17 0    83  54.88 51.8160  900   900 0.000 0 0 0  0.0  0  0  0 0 0   0.000
 43   18 0   168  55.01 51.0140  900   900 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 43   19 0    88  51.21 48.3990  900   900 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 43   20 0    92  50.77 47.8050  900   900 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 43   21 0    95  50.05 47.1530  900   900 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 43   22 0    68  48.33 45.4910  900   900 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 56    0 2   769   0.00  0.0206  300    016.245 0 0 0  0.0 20  0  2 2 0   0.000
 43   23 0    52  47.90 45.0800  900   900 0.000 0 0 0  0.0  0  0  0 1 0   0.000
600   14 0   149  48.18 44.4550 1140  1140 0.000 0 0 0  0.0  0  0  0 0 0   0.000
 57    0 2   650   0.00  0.0147  300     0 8.153 0 0 0  0.0 16 32  2 1 0   1.000
700    5 0     3  46.25 44.4720  375     0 0.000 1
800 60015     3  43.44 43.5700  450   450 0.000 0 0 0  0.0  0  0  0 0 0   0.000
500   16 0    94  46.25 44.3250  375   375 0.000 0 0 0  0.0  0  0  0 0 0   0.000
500   17 0   113  46.94 43.7520  450   450 0.000 0 0 0  0.0  0  0  0 0 0   0.000
 58    0 2   383   0.00  0.0447  370     0 3.113 0 0 0  0.0 36  0  2 2 0   1.000
500   18 0    58  48.04 42.9510  600   600 0.000 0 0 0  0.0  0  0  0 0 0   0.000
500   19 0    97  47.86 42.5400  600   600 0.000 0 0 0  0.0  0  0  0 0 0   0.000
 59    0 2   405   0.00  0.0660  225    017.801 0 0 0  0.0 58  0  2 3 0  18.000
 60    0 2   754   0.00  0.0154  375    019.455 0 0 0  0.0 40 12  2 2 0  18.000
 61    0 2   542   0.00  0.0226  225     0 3.536 0 0 0  0.0 24  0  2 2 0   8.000
 59    1 0    58  89.75 86.9530  450   450 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 62    0 2   479   0.00  0.0113  300     0 3.599 0 0 0  0.0 14  0  2 1 0   2.000
 59    2 0    66  85.26 83.1160  450   450 0.000 0 0 0  0.0  0  0  0 1 0   0.000
 63    0 2   111   0.00  0.0217  225     0 0.982 0 0 0  0.0 48  0  2 1 0   2.000
 64    0 2   483   0.00  0.0305  225     0 5.010 0 0 0  0.0 32  0  2 2 0   3.000
 59    3 0   192  86.30 83.0020  450   450 0.000 0 1 0  0.0  0  0  0 1 0   0.000
 65    0 2   951   0.00  0.0032  300    010.334 0 0 0  0.0 58  0  2 2 0   1.000
 66    0 2   191   0.00  0.0270  225     0 2.335 0 0 0  0.0 24  0  2 2 0   1.000
 59    4 0   815  84.61 82.1130  450   450 0.000 0 5 0  0.0  0  0  0 3 0   0.000
 67    0 2   406   0.00  0.0364  600     0 6.226 0 0 0  0.0 46  0  2 2 0   1.000
500   20 0    88  47.62 41.8760  750   750 0.000 0 0 0  0.0  0  0  0 0 0   0.000
 68    0 2   695   0.00  0.0299  300     0 7.145 0 0 0  0.0 39 20  3 2 0   4.000
500   21 3    30  45.40 41.0800  600   600 0.000 1 0 0  0.0  0  0  0 0 0   0.000
-2
500   22 3 40.69
600   15 0    98  46.03 43.3400 1140  1140 0.000 0 0 0  0.0  0  0  0 0 0   0.000
600   16 0    90  46.95 42.8860 1140  1140 0.000 0 0 0  0.0  0  0  0 0 0   0.000
 69    0 2   383   0.00  0.0447  900     0 3.113 0 0 0  0.0 36  0  2 2 0   0.000
600   17 0    70  47.57 42.3330 1140  1140 0.000 0 0 0  0.0  0  0  0 0 0   0.000
600   18 0    87  47.86 41.4220 1140  1140 0.000 0 0 0  0.0  0  0  0 0 0   0.000
600   19 0    78  46.12 40.5110 1080  1080 0.000 0 0 0  0.0  0  0  0 0 0   0.000
 70    0 2   889   0.00  0.0704  500    023.477 0 0 0  0.0 40 30  5 3 0   5.000
600   20 0    38  45.40 39.6020 1200  1200 0.000 0 0 0  0.0  0  0  0 0 0   0.000
-1
600   21 3 39.17
```

APPENDIX A(xii)

SAMPLE OF PCD INPUT DATA FILE

```
BRUCEFIELD HOUSE RAINGAUGE
99/99/99
00:00
0.9519
0
60
60
2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0
2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0
2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0
2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0
2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0
2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0  2.0
```

APPENDIX A(xiii)
SAMPLE OF PARALLEL PIPE MODEL (DUPPERS) SIMULATION OUTPUT
(DUPOUT.DAT)

```
  DDDDDDD   UUUU   UUUU PPPPPPP    PPPPPPP    EEEEEEEE   RRRRRRR      SSSSS
   DD   DD   UU      UU   PP   PP    PP   PP    EE     EE   RR   RR    SS   SS
   DD    DD  UU      UU   PP    PP   PP    PP   EE     EE   RR    RR  SS     SS
   DD    DD  UU      UU   PP    PP   PP    PP   EE          RR    RR  SS
   DD    DD  UU      UU   PP    PP   PP    PP   EE  E       RR    RR  SS
   DD    DD  UU      UU   PP   PP    PP   PP    EEEEE       RR   RR    SSSSSSS
   DD    DD  UU      UU   PPPPPP     PPPPPP     EE  E       RRRRRR           SS
   DD    DD  UU      UU   PP         PP         EE          RR R             SS
   DD    DD  UU      UU   PP         PP         EE          RR  R            SS
   DD    DD  UU      UU   PP         PP         EE    EE    RR   R    SS     SS
   DD   DD    UU    UU    PP         PP         EE    EE    RR    R   SS    SS
  DDDDDDD      UUUUU     PPPP       PPPP       EEEEEEEE   RRRR    R   SSSSSS
```

DUNFERMLINE PARALLEL PIPES RESEARCH SIMULATION

```
RAINFALL DATA USED IN SIMULATION
DATAFILE   PARPCD18
RAIN DATE  99/99/99
START TIME 00:00
   0.1   1.9   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0
   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0
   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0
   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0
   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0
   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0   2.0
   1.9   0.1
```

****

****

COMBINED SEWERAGE SYSTEM SIMULATION

```
RESULTS OF SIMULATION RUN       DATE   12-APR-89
PRINTED FROM FILE DUCOUT.DAT
SYSTEM NAME  BOTHWELL ST CATCHMENT TOTAL INFLOW -- PARALLEL PIPE VERSION
SYSTEM DATA FILE.... BOTHPR1
RECORDED RAIN FILE.. PARPCD18
RESULTS USING RECORDED STORM OF 99/99/99
URBAN CATCHMENT WETNESS INDEX = 132.62    PERCENTAGE RUNOFF = 38.344 %  S.P.R. =  0.000 %
PRPAV = 72.882    PRPRV =  9.607
*** DEPRESSION STORAGE NOT SATISFIED ***
STORM LENGTH =   60MINS
SURCHARGING INFORMATION
```

****

MAX FLOWS IN EACH PIPE

END ORDER NO	BRANCH NO	PIPE NO	FULL BORE FLOW (L/S)	MAX FLOW (L/S)	MAX LEVEL (M)
1	1	0	121.52	0.00	0.0000
2	1	1	127.61	16.99	0.0000
3	1	2	218.10	16.99	0.0000
4	2	0	75.11	0.00	0.0000
5	1	3	102.79	24.82	0.0000
6	1	4	252.48	24.82	0.0000
7	3	0	92.28	0.20	0.0000
8	1	5	101.93	25.02	0.0000
9	1	6	142.00	25.02	0.0000
10	1	7	196.83	25.02	0.0000

11	4	0	73.02	0.20	0.0000
12	1	8	134.46	25.22	0.0000
13	1	9	489.66	25.22	0.0000
14	5	0	192.46	0.00	0.0000
15	6	0	52.74	0.00	0.0000
16	6	1	273.60	3.23	0.0000
17	1	10	237.91	36.04	0.0000
18	7	0	263.20	0.00	0.0000
19	7	1	180.36	49.17	0.0000
20	7	2	191.18	49.16	0.0000
21	7	3	115.35	49.14	0.0000
22	7	4	147.94	49.50	0.0000
23	7	5	303.94	49.21	0.0000
24	8	0	115.61	0.00	0.0000
25	9	0	133.23	0.00	0.0000
26	7	6	77.70	52.89	0.0000
27	10	0	66.68	0.00	0.0000
28	7	7	239.49	53.39	0.0000
29	11	0	35.17	0.00	0.0000
30	7	8	222.04	60.05	0.0000
31	12	0	34.07	0.00	0.0000
32	7	9	221.33	64.04	0.0000
33	13	0	16.03	0.00	0.0000
34	14	0	25.69	0.00	0.0000
35	7	10	171.93	67.56	0.0000
36	15	0	33.36	0.00	0.0000
37	7	11	209.57	69.26	0.0000
38	16	0	14.98	0.00	0.0000
39	7	12	107.33	70.02	0.0000
40	17	0	70.47	0.00	0.0000
41	7	13	379.28	75.95	0.0000
42	7	14	199.89	78.98	0.0000
43	18	0	78.08	0.00	0.0000
44	7	15	945.83	89.57	0.0000
45	7	16	140.53	89.57	0.0000
46	7	17	130.38	89.57	0.0000
47	7	18	116.56	89.57	0.0000
48	7	19	85.70	88.56	0.0000
49	7	20	89.00	83.08	0.0000
50	1	11	802.20	119.13	0.0000
51	1	12	1044.86	119.13	0.0000
52	19	0	17.60	0.00	0.0000
53	19	1	12.75	0.00	0.0000
54	19	2	29.79	0.10	0.0000
55	19	3	41.78	0.20	0.0000
56	19	4	34.09	0.30	0.0000
57	19	5	63.23	0.40	0.0000
58	19	6	41.79	0.50	0.0000
59	19	7	49.31	0.60	0.0000
60	1	13	865.06	119.73	0.0000
61	1	14	676.69	119.73	0.0000
62	20	0	72.95	0.20	0.0000
63	20	1	68.38	0.40	0.0000
64	1	15	478.94	120.13	0.0000
65	21	0	13.51	0.00	0.0000
66	21	1	11.18	0.10	0.0000
67	21	2	50.47	0.20	0.0000
68	21	3	17.97	0.30	0.0000
69	21	4	44.06	0.40	0.0000
70	21	5	48.30	0.50	0.0000
71	21	6	68.79	0.60	0.0000
72	21	7	177.39	0.70	0.0000
73	21	8	185.99	0.80	0.0000
74	1	16	655.73	120.93	0.0000
75	22	0	34.22	0.20	0.0000
76	22	1	34.22	0.40	0.0000
77	22	2	34.22	0.70	0.0000
78	22	3	34.22	1.00	0.0000
79	1	17	397.96	121.93	0.0000
80	1	18	443.45	121.93	0.0000
81	1	19	382.02	121.93	0.0000
82	23	0	348.62	0.20	0.0000
83	24	0	136.84	0.00	0.0000
84	1	20	521.93	151.23	0.0000
85	1	21	570.06	151.23	0.0000
86	25	0	321.04	0.80	0.0000
87	1	22	553.66	152.03	0.0000
88	1	23	1507.92	152.00	0.0000
89	700	1	0.00	152.00	0.0000
90	500	0	286.63	152.00	0.0000
91	26	0	527.94	0.40	0.0000
92	26	1	499.04	0.40	0.0000
93	27	0	527.20	0.40	0.0000
94	26	2	181.70	0.80	0.0000

A(xiii)

95	28	0	253.76	0.40	0.0000
96	26	3	969.89	1.20	0.0000
97	500	1	234.82	153.17	59.2401
98	500	2	219.64	155.35	59.0089
99	29	0	322.43	0.00	0.0000
100	500	3	193.36	186.42	58.5265
101	30	0	98.73	0.00	0.0000
102	500	4	194.94	188.40	57.9217
103	31	0	229.08	0.00	0.0000
104	500	5	139.86	212.79	57.1835
105	800	1	4006.88	0.00	0.0000
106	600	0	1269.55	0.01	0.0000
107	600	1	1262.44	0.02	0.0000
108	600	2	1341.84	0.02	0.0000
109	600	3	1366.09	0.10	0.0000
110	600	4	1539.52	0.04	0.0000
111	600	5	1477.93	0.02	0.0000
112	32	0	181.94	0.00	0.0000
113	700	2	0.00	213.59	0.0000
114	500	6	219.66	213.54	56.8384
115	500	7	164.92	211.87	55.9164
116	800	2	799.46	0.00	0.0000
117	600	6	1396.50	0.06	0.0000
118	600	7	1987.61	2.72	0.0000
119	600	8	1509.03	10.30	0.0000
120	700	3	0.00	211.87	0.0000
121	500	8	456.46	212.18	56.2047
122	33	0	230.80	0.00	0.0000
123	500	9	247.97	216.87	55.9543
124	500	10	120.50	216.82	54.0227
125	500	11	126.44	176.82	51.3800
126	34	0	77.98	0.80	0.0000
127	35	0	30.87	0.80	0.0000
128	36	0	168.73	0.00	0.0000
129	500	12	137.42	190.89	49.8882
130	37	0	64.50	0.00	0.0000
131	500	13	157.47	192.67	46.3446
132	800	3	626.64	0.00	0.0000
133	600	9	3101.62	1.85	0.0000
134	600	10	1846.16	0.04	0.0000
135	600	11	1900.10	61.21	0.0000
136	600	12	1888.30	25.79	0.0000
137	600	13	2252.84	6.25	0.0000
138	38	0	156.63	0.00	0.0000
139	39	0	28.08	0.00	0.0000
140	38	1	161.17	4.85	0.0000
141	38	2	159.43	4.85	0.0000
142	40	0	164.34	0.00	0.0000
143	41	0	56.79	0.00	0.0000
144	38	3	144.47	17.26	0.0000
145	38	4	142.27	17.25	0.0000
146	42	0	16.46	0.00	0.0000
147	38	5	144.85	22.43	0.0000
148	38	6	146.25	22.43	0.0000
149	38	7	153.20	22.43	0.0000
150	700	4	0.00	229.06	0.0000
151	500	14	219.83	229.06	45.8750
152	500	15	146.53	213.44	0.0000
153	43	0	400.70	0.00	0.0000
154	43	1	206.20	12.15	0.0000
155	43	2	201.39	12.15	0.0000
156	44	0	38.67	0.00	0.0000
157	43	3	217.17	12.14	0.0000
158	43	4	326.42	12.14	0.0000
159	43	5	298.94	12.19	0.0000
160	45	0	651.10	0.00	0.0000
161	43	6	284.14	18.99	0.0000
162	46	0	45.43	0.00	0.0000
163	43	7	124.87	18.99	0.0000
164	47	0	15.35	0.00	0.0000
165	48	0	307.53	0.00	0.0000
166	43	8	324.31	42.41	0.0000
167	43	9	272.48	42.41	0.0000
168	49	0	22.90	0.00	0.0000
169	43	10	1299.23	42.45	0.0000
170	43	11	1103.83	42.54	0.0000
171	43	12	1714.61	42.45	0.0000
172	50	0	190.60	0.00	0.0000
173	51	0	122.13	0.00	0.0000
174	43	13	2179.69	61.52	0.0000
175	43	14	1325.10	61.52	0.0000
176	52	0	121.63	0.00	0.0000

177	43	15	1688.70	61.52	0.0000
178	53	0	404.43	0.00	0.0000
179	54	0	167.46	0.00	0.0000
180	43	16	1915.33	80.66	0.0000
181	55	0	600.65	0.00	0.0000
182	43	17	1440.36	89.14	0.0000
183	43	18	1828.63	89.14	0.0000
184	43	19	1203.53	89.15	0.0000
185	43	20	1233.26	89.17	0.0000
186	43	21	1938.76	89.15	0.0000
187	43	22	1138.76	89.14	0.0000
188	56	0	111.87	0.00	0.0000
189	43	23	1606.64	110.10	0.0000
190	800	4	1342.72	0.00	0.0000
191	600	14	2374.58	136.92	0.0000
192	57	0	94.46	0.00	0.0000
193	700	5	0.00	135.24	0.0000
194	500	16	110.26	131.18	44.5991
195	500	17	193.94	131.18	0.0000
196	58	0	289.75	0.00	0.0000
197	500	18	418.59	125.99	0.0000
198	500	19	411.40	129.07	0.0000
199	59	0	92.55	0.00	0.0000
200	60	0	177.01	0.00	0.0000
201	61	0	54.10	0.00	0.0000
202	59	1	593.47	136.01	0.0000
203	62	0	82.79	0.00	0.0000
204	59	2	95.49	129.18	83.2274
205	63	0	53.01	0.00	0.0000
206	64	0	62.87	0.00	0.0000
207	59	3	156.65	144.99	0.0000
208	65	0	43.95	0.00	0.0000
209	66	0	59.14	0.00	0.0000
210	59	4	512.62	176.19	0.0000
211	67	0	956.80	0.00	0.0000
212	500	20	857.77	248.94	0.0000
213	68	0	134.82	0.00	0.0000
214	500	21	915.32	264.64	0.0000
215	500	22	0.00	263.42	0.0000
216	800	5	480.22	0.00	0.0000
217	600	15	1867.68	138.66	0.0000
218	600	16	2151.42	115.86	0.0000
219	69	0	3125.56	0.00	0.0000
220	600	17	3132.49	133.18	0.0000
221	600	18	2809.51	141.27	0.0000
222	600	19	2567.88	135.64	0.0000
223	70	0	817.24	0.00	0.0000
224	600	20	3354.23	180.23	0.0000
225	600	21	0.00	187.85	0.0000

A ZERO VALUE IN MAX LEVEL COLUMN INDICATES NO SURCHARGING

PIPE NO 500.22 IS A DUMMY PIPE
PIPE NO 600.21 IS A DUMMY PIPE

****

****

FLOW AND LEVEL HYDROGRAPHS AT SELECTED NODES
==== === ===== ========== == ======== =====

FLOWRATES IN LITRES/SEC AT 1 MIN INTERVALS
LEVELS IN METRES AT 1 MINUTE INTERVALS

DISCHARGE HYDROGRAPH --- LOCATION IS BRANCH 1 PIPE 23

20.90	20.90	20.90	20.90	20.90	20.90	20.90	22.62	25.69	29.63
34.71	40.77	47.91	55.99	64.56	73.08	81.26	88.93	96.11	102.78
108.99	114.79	120.14	125.18	129.99	134.51	138.59	142.14	145.12	147.58
149.54	150.30	150.65	150.89	151.08	151.23	151.36	151.47	151.55	151.63
151.69	151.73	151.78	151.81	151.84	151.86	151.88	151.90	151.91	151.92
151.93	151.94	151.94	151.95	151.95	151.96	151.96	151.96	151.96	151.97
151.97	151.97	152.00	151.58	150.76	149.70	148.28	146.60	144.53	141.76
137.26	130.57	122.25	113.12	103.99	95.65	88.27	81.82	75.98	70.67
65.68	61.01	56.66	52.44	48.29	44.34	40.73	37.71	35.29	33.35
31.80	30.50	29.42	28.49	27.71	26.91	26.10	25.35	24.68	24.08
23.53	23.09	22.75	22.43	22.11	21.83	21.56	21.33	21.18	

TOTAL VOLUME OF RUNOFF = 613.4379 CUBIC METRES
Max number of surcharged pipes = 16

A(xiii)

```
LEVEL HYDROGRAPH --- LOCATION IS BRANCH     1    PIPE   23
    0.0414    0.0392    0.0414    0.0434    0.0434    0.0454    0.0473    0.0507    0.0539    0.0584
    0.0625    0.0676    0.0734    0.0788    0.0847    0.0894    0.0946    0.0988    0.1028    0.1059
    0.1090    0.1118    0.1147    0.1168    0.1189    0.1209    0.1229    0.1249    0.1262    0.1268
    0.1271    0.1277    0.1279    0.1281    0.1283    0.1284    0.1285    0.1287    0.1289    0.1290
    0.1291    0.1292    0.1293    0.1294    0.1295    0.1295    0.1296    0.1297    0.1298    0.1298
    0.1298    0.1299    0.1300    0.1301    0.1301    0.1302    0.1303    0.1303    0.1304    0.1305
    0.1306    0.1307    0.1307    0.1303    0.1293    0.1281    0.1274    0.1268    0.1262    0.1249
    0.1229    0.1202    0.1161    0.1118    0.1075    0.1028    0.0988    0.0955    0.0920    0.0894
    0.0857    0.0828    0.0798    0.0777    0.0745    0.0712    0.0688    0.0664    0.0638    0.0625
    0.0612    0.0598    0.0584    0.0569    0.0569    0.0555    0.0555    0.0539    0.0539    0.0524
    0.0524    0.0524    0.0524    0.0507    0.0507    0.0507    0.0507    0.0507    0.0507
MAXIMUM LEVEL IS      0.1307 METRES


DISCHARGE HYDROGRAPH --- LOCATION IS BRANCH   700    PIPE    1
    20.90     20.90     20.90     20.90     20.90     20.90     20.90     21.62     24.64     28.24
    33.08     38.87     45.71     53.60     62.12     70.74     79.05     86.90     94.23    101.06
   107.40    113.33    118.80    123.92    128.80    133.41    137.62    141.31    144.43    147.01
   149.19    150.17    150.57    150.83    151.03    151.19    151.33    151.44    151.53    151.61
   151.67    151.72    151.77    151.80    151.83    151.86    151.88    151.89    151.91    151.92
   151.93    151.94    151.94    151.95    151.95    151.96    151.96    151.96    151.96    151.97
   151.97    151.97    151.98    152.00    151.06    150.36    148.83    147.44    145.32    142.94
   138.87    132.71    124.77    115.70    106.46     97.79     90.16     83.44     77.47     71.99
    66.95     62.16     57.75     53.49     49.32     45.28     41.64     38.37     35.90     33.77
    32.21     30.78     29.73     28.69     27.93     27.13     26.30     25.53     24.83     24.22
    23.65     23.19     22.83     22.51     22.18     21.90     21.63     21.38     21.21
TOTAL VOLUME OF RUNOFF =      613.4689 CUBIC METRES
Max number of surcharged pipes =  16


LEVEL HYDROGRAPH --- LOCATION IS BRANCH   700    PIPE    1
    0.0309    0.0477    0.0562    0.0679    0.0619    0.0687    0.0776    0.0801    0.0827    0.0877
    0.0888    0.0920    0.0935    0.0953    0.0968    0.0977    0.0994    0.1021    0.1046    0.1061
    0.1085    0.1145    0.1193    0.1287    0.1490    0.1682    0.1858    0.2011    0.2141    0.2249
    0.2340    0.2380    0.2397    0.2408    0.2416    0.2423    0.2429    0.2433    0.2437    0.2440
    0.2443    0.2445    0.2447    0.2448    0.2450    0.2451    0.2451    0.2452    0.2453    0.2453
    0.2454    0.2454    0.2454    0.2454    0.2455    0.2455    0.2455    0.2455    0.2455    0.2455
    0.2455    0.2455    0.2456    0.2457    0.2417    0.2388    0.2325    0.2267    0.2178    0.2079
    0.1910    0.1653    0.1322    0.0944    0.0559    0.0198    0.0195    0.0191    0.0190    0.0188
    0.0186    0.0183    0.0181    0.0178    0.0175    0.0173    0.0172    0.0170    0.0168    0.0165
    0.0163    0.0161    0.0159    0.0156    0.0154    0.0152    0.0150    0.0148    0.0145    0.0143
    0.0141    0.0139    0.0136    0.0134    0.0132    0.0130    0.0128    0.0126    0.0120
MAXIMUM LEVEL IS      0.2457 METRES


DISCHARGE HYDROGRAPH --- LOCATION IS BRANCH   500    PIPE   22
   103.20    103.20    103.20    103.20    103.20    103.20    104.84    117.55    128.65    143.52
   161.08    179.20    196.75    214.91    230.03    211.50    263.42    245.76    168.64    172.73
   243.62    189.02    185.86    190.12    191.49    192.99    196.70    197.85    198.80    218.43
   221.24    223.89    224.74    225.36    225.95    226.52    227.09    227.64    228.19    228.73
   229.26    229.78    230.30    230.80    231.30    231.79    232.27    232.74    233.20    233.64
   234.06    234.47    234.86    235.23    235.58    235.91    236.22    236.51    236.78    237.02
   237.25    237.46    237.71    235.60    232.97    228.32    223.62    199.42    194.38    184.72
   174.69    165.91    158.02    152.52    146.31    143.09    124.99    115.42    109.71    105.19
   103.20    103.20    103.20    116.64    215.33    204.31    196.35    189.30    179.29    172.52
   165.57    159.32    152.52    146.47    140.89    136.39    132.54    129.39    126.71    124.50
   122.57    120.88    119.35    117.85    116.40    115.03    113.78    112.64    111.60
TOTAL VOLUME OF RUNOFF =     1184.9243 CUBIC METRES
Max number of surcharged pipes =  16


LEVEL HYDROGRAPH --- LOCATION IS BRANCH   500    PIPE   22
    0.1052    0.1067    0.1189    0.1255    0.1300    0.1319    0.1385    0.1466    0.1537    0.1622
    0.1722    0.1813    0.1906    0.1990    0.2081    0.2051    0.2206    0.1999    0.1747    0.1775
    0.2202    0.1837    0.1841    0.1860    0.1865    0.1879    0.1892    0.1897    0.1910    0.1990
    0.2003    0.2017    0.2021    0.2021    0.2026    0.2030    0.2030    0.2034    0.2034    0.2038
    0.2042    0.2042    0.2047    0.2047    0.2051    0.2051    0.2056    0.2056    0.2060    0.2060
    0.2064    0.2064    0.2068    0.2068    0.2068    0.2072    0.2072    0.2072    0.2077    0.2077
    0.2077    0.2077    0.2081    0.2068    0.2056    0.2034    0.1999    0.1897    0.1869    0.1818
    0.1766    0.1722    0.1688    0.1652    0.1622    0.1585    0.1483    0.1432    0.1397    0.1367
    0.1343    0.1325    0.1306    0.1601    0.1969    0.1919    0.1883    0.1841    0.1795    0.1761
    0.1722    0.1688    0.1652    0.1616    0.1585    0.1564    0.1543    0.1521    0.1505    0.1493
    0.1483    0.1471    0.1460    0.1454    0.1443    0.1438    0.1426    0.1420    0.1414
MAXIMUM LEVEL IS      0.2206 METRES
```

```
DISCHARGE HYDROGRAPH --- LOCATION IS BRANCH   600    PIPE   21
      5.02      5.02      5.02      8.60     12.40     19.73     26.18     36.25     48.30     61.95
     74.56     85.98     96.34    106.35    116.27    148.27    134.06    152.27    143.13    147.98
    171.38    156.09    159.13    161.72    163.86    166.64    169.26    168.34    172.91    187.85
    170.98    171.30    171.73    172.08    172.37    172.60    172.79    172.94    173.06    173.16
    173.24    173.30    173.36    173.40    173.43    173.46    173.48    173.50    173.51    173.52
    173.53    173.54    173.55    173.55    173.56    173.56    173.56    173.56    173.56    173.57
    173.57    173.61    174.27    167.64    164.28    152.89    147.51    133.29    120.46    103.64
     90.97     79.69     70.65     61.45     53.70     46.81     41.41     36.59     32.27     28.43
     25.18     22.52     20.34     40.30     17.55     15.78     14.43     13.53     12.48     11.71
     10.59      9.58      8.62      8.08      7.51      7.06      6.47      6.18      5.95      5.82
      5.66      5.56      5.43      5.35      5.25      5.15      5.09      5.05      5.03
TOTAL VOLUME OF RUNOFF =      640.4432 CUBIC METRES
Max number of surcharged pipes =  16


LEVEL HYDROGRAPH --- LOCATION IS BRANCH   600    PIPE   21
    0.0311    0.0311    0.0311    0.0422    0.0505    0.0689    0.0785    0.0908    0.1014    0.1139
    0.1250    0.1352    0.1423    0.1490    0.1554    0.1694    0.1675    0.1714    0.1694    0.1732
    0.1806    0.1769    0.1788    0.1806    0.1806    0.1841    0.1841    0.1841    0.1892    0.1892
    0.1841    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859
    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859
    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859
    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859    0.1859
    0.1859    0.1859    0.1859    0.1823    0.1806    0.1751    0.1714    0.1636    0.1554    0.1446
    0.1352    0.1277    0.1196    0.1110    0.1048    0.0980    0.0946    0.0869    0.0828    0.0785
    0.0739    0.0689    0.0635    0.0980    0.0635    0.0575    0.0575    0.0505    0.0505    0.0505
    0.0505    0.0422    0.0422    0.0422    0.0422    0.0422    0.0311    0.0311    0.0311    0.0311
    0.0311    0.0311    0.0311    0.0311    0.0311    0.0311    0.0311    0.0311    0.0311
MAXIMUM LEVEL IS     0.1892 METRES


SURFACE FLOODING INFORMATION
------- -------- -----------
VOLUME ON PIPE   500   11 =      103.8 CUBIC METRES
**********************************************************************************************************************

**********************************************************************************************************************
```

APPENDIX A(xiv)

SAMPLE OF DUPPERS CHECKING OUTPUT FILE
(DUPCHK.DAT)

****

****

DUNFERMLINE PARALLEL PIPE RESEARCH SIMULATION MODEL

BY O.W.K AU-YEUNG

SEWER SYSTEM LAYOUT AND DETAILS FOR CHECKING

Printed from file DUSCHK.DAT
SYSTEM NAME BOTHWELL ST CATCHMENT TOTAL INFLOW -- PARALLEL PIPE VERSION
SYSTEM CONTAINS 224 PIPES / ANCILLIARIES

DATA LINE NUMBER	END ORDER NUMBER	BRANCH NO	PIPE NO	M.H. DOWN STREAM
1	1	1	0	2
2	2	1	1	3
3	3	1	2	5
4	4	2	0	5
5	5	1	3	6
6	6	1	4	8
7	7	3	0	8
8	8	1	5	9
9	9	1	6	10
10	10	1	7	12
11	11	4	0	12
12	12	1	8	13
13	13	1	9	17
14	14	5	0	17
15	15	6	0	16
16	16	6	1	17
17	17	1	10	50
18	18	7	0	19
19	19	7	1	20
20	20	7	2	21
21	21	7	3	22
22	22	7	4	23
23	23	7	5	26
24	24	8	0	26
25	25	9	0	26
26	26	7	6	28
27	27	10	0	28
28	28	7	7	30
29	29	11	0	30
30	30	7	8	32
31	31	12	0	32
32	32	7	9	35
33	33	13	0	35
34	34	14	0	35
35	35	7	10	37
36	36	15	0	37
37	37	7	11	39
38	38	16	0	39
39	39	7	12	41
40	40	17	0	41
41	41	7	13	42
42	42	7	14	44
43	43	18	0	44
44	44	7	15	45
45	45	7	16	46
46	46	7	17	47
47	47	7	18	48
48	48	7	19	49
49	49	7	20	50
50	50	1	11	51
51	51	1	12	60
52	52	19	0	53
53	53	19	1	54
54	54	19	2	55

55	55	19	3	56
56	56	19	4	57
57	57	19	5	58
58	58	19	6	59
59	59	19	7	60
60	60	1	13	61
61	61	1	14	64
62	62	20	0	63
63	63	20	1	64
64	64	1	15	74
65	65	21	0	66
66	66	21	1	67
67	67	21	2	68
68	68	21	3	69
69	69	21	4	70
70	70	21	5	71
71	71	21	6	72
72	72	21	7	73
73	73	21	8	74
74	74	1	16	79
75	75	22	0	76
76	76	22	1	77
77	77	22	2	78
78	78	22	3	79
79	79	1	17	80
80	80	1	18	81
81	81	1	19	84
82	82	23	0	84
83	83	24	0	84
84	84	1	20	85
85	85	1	21	87
86	86	25	0	87
87	87	1	22	88
88	88	1	23	89
89	89	700	1	90
90	90	500	0	97
91	91	26	0	92
92	92	26	1	94
93	93	27	0	94
94	94	26	2	96
95	95	28	0	96
96	96	26	3	97
97	97	500	1	98
98	98	500	2	100
99	99	29	0	100
100	100	500	3	102
101	101	30	0	102
102	102	500	4	104
103	103	31	0	104
104	104	500	5	113
105	105	800	1	106
106	106	600	0	107
107	107	600	1	108
108	108	600	2	109
109	109	600	3	110
110	110	600	4	111
111	111	600	5	117
112	112	32	0	113
113	113	700	2	114
114	114	500	6	115
115	115	500	7	120
116	116	800	2	117
117	117	600	6	118
118	118	600	7	119
119	119	600	8	133
120	120	700	3	121
121	121	500	8	123
122	122	33	0	123
123	123	500	9	124
124	124	500	10	125
125	125	500	11	129
126	126	34	0	129
127	127	35	0	129
128	128	36	0	129
129	129	500	12	131
130	130	37	0	131
131	131	500	13	150
132	132	800	3	133
133	133	600	9	134
134	134	600	10	135
135	135	600	11	136
136	136	600	12	137
137	137	600	13	191
138	138	38	0	140

A(xiv)

139	139	39	0	140
140	140	38	1	141
141	141	38	2	144
142	142	40	0	144
143	143	41	0	144
144	144	38	3	145
145	145	38	4	147
146	146	42	0	147
147	147	38	5	148
148	148	38	6	149
149	149	38	7	150
150	150	700	4	151
151	151	500	14	152
152	152	500	15	193
153	153	43	0	154
154	154	43	1	155
155	155	43	2	157
156	156	44	0	157
157	157	43	3	158
158	158	43	4	159
159	159	43	5	161
160	160	45	0	161
161	161	43	6	163
162	162	46	0	163
163	163	43	7	166
164	164	47	0	166
165	165	48	0	166
166	166	43	8	167
167	167	43	9	169
168	168	49	0	169
169	169	43	10	170
170	170	43	11	171
171	171	43	12	174
172	172	50	0	174
173	173	51	0	174
174	174	43	13	175
175	175	43	14	177
176	176	52	0	177
177	177	43	15	180
178	178	53	0	180
179	179	54	0	180
180	180	43	16	182
181	181	55	0	182
182	182	43	17	183
183	183	43	18	184
184	184	43	19	185
185	185	43	20	186
186	186	43	21	187
187	187	43	22	189
188	188	56	0	189
189	189	43	23	191
190	190	800	4	191
191	191	600	14	217
192	192	57	0	193
193	193	700	5	194
194	194	500	16	195
195	195	500	17	197
196	196	58	0	197
197	197	500	18	198
198	198	500	19	212
199	199	59	0	202
200	200	60	0	202
201	201	61	0	202
202	202	59	1	204
203	203	62	0	204
204	204	59	2	207
205	205	63	0	207
206	206	64	0	207
207	207	59	3	210
208	208	65	0	210
209	209	66	0	210
210	210	59	4	212
211	211	67	0	212
212	212	500	20	214
213	213	68	0	214
214	214	500	21	215
216	216	800	5	217
217	217	600	15	218
218	218	600	16	220
219	219	69	0	220
220	220	600	17	221
221	221	600	18	222
222	222	600	19	224
223	223	70	0	224
224	224	600	20	225

****

SYSTEM NAME BOTHWELL ST CATCHMENT TOTAL INFLOW -- PARALLEL PIPE VERSION

CATCHMENT DETAILS

GLOBAL VARIABLES
PIPE ROUGHNESS HEIGHT : 6.000 MM
GLOBAL DRY WEATHER FLOW : .0000000 LPS/METRE. STANDARD ANNUAL AVERAGE RAINFALL : 780 MM
PERCENTAGE IMPERMEABILITY : 10.000 SOIL TYPE : 0.45

BN NO	PIPE NO	LEN M	TOP INV LEV M	SLOPE	D1 MM	D2 MM	ROUGH NESS MM	CONT AREA HA	% IMP	% PITCH ROOF	% FLOOD AREA	CUM DWF LIT/S	TOP GL M	PAPG	EXTRA MH
1	0 *														
1	1	130.	94.23	0.0031	450	450	6.0	0.000	10	5	2	4.00	97.98	-2	0
1	2	25.	93.83	0.0090	450	450	6.0	0.000	10	5	2	4.00	96.88	-2	0
2	0 *														
1	3	25.	93.61	0.0020	450	450	6.0	0.000	10	5	2	4.30	96.40	-2	0
1	4	20.	93.56	0.0120	450	450	6.0	0.000	10	5	2	4.30	95.84	-2	0
3	0	151.	103.38	0.0666	225	225	6.0	0.000	10	5	2	0.20	105.11	-2	0
1	5	90.	93.32	0.0020	450	450	6.0	0.000	10	5	2	4.50	94.89	-2	0
1	6	57.	93.14	0.0038	450	450	6.0	0.000	10	5	2	4.50	94.59	-2	0
1	7	10.	92.92	0.0073	450	450	6.0	0.000	10	5	2	4.50	95.11	-2	0
4	0	273.	104.25	0.0418	225	225	6.0	0.000	10	5	2	0.20	106.52	-2	0
1	8	82.	92.85	0.0034	450	450	6.0	0.000	10	5	2	4.70	95.29	-2	0
1	9	217.	92.57	0.0451	450	450	6.0	0.000	10	5	2	4.70	96.51	-2	0
5	0 *														
6	0 *														
6	1	20.	83.54	0.0374	375	375	6.0	0.000	10	5	2	0.60	84.38	-2	0
1	10	93.	82.79	0.0283	375	375	6.0	0.000	10	5	2	6.10	84.32	-2	0
7	0 *														
7	1	230.	98.69	0.0163	375	375	6.0	0.000	10	5	2	2.00	100.12	-2	0
7	2	75.	94.94	0.0183	375	375	6.0	0.000	10	5	2	2.00	98.60	-2	0
7	3	90.	93.57	0.0221	300	300	6.0	0.000	10	5	2	2.00	97.12	-2	0
7	4	90.	91.59	0.0363	300	300	6.0	0.000	10	5	2	2.00	95.29	-2	0
7	5	20.	88.32	0.1530	300	300	6.0	0.000	10	5	2	2.00	92.33	-2	0
8	0 *														
9	0 *														
7	6	85.	85.26	0.0030	375	375	6.0	0.430	9	5	2	3.00	88.41	-2	0
10	0 *														
7	7	9.	85.00	0.0287	375	375	6.0	0.000	10	5	2	3.20	86.18	-2	0
11	0 *														
7	8	63.	84.74	0.0093	450	450	6.0	0.000	10	5	2	4.00	86.51	-2	0
12	0 *														
7	9	75.	84.16	0.0092	450	450	6.0	0.000	10	5	2	4.60	86.46	-2	0
13	0 *														
14	0 *														
7	10	47.	83.47	0.0056	450	450	6.0	0.000	10	5	2	4.90	85.89	-2	0
15	0 *														
7	11	84.	83.20	0.0083	450	450	6.0	0.490	3	14	2	5.10	85.18	-2	0
16	0 *														
7	12	78.	82.51	0.0022	450	450	6.0	0.000	10	5	2	5.20	84.94	-2	0
17	0 *														
7	13	22.	82.34	0.0270	450	450	6.0	0.000	10	5	2	6.00	84.42	-2	0
7	14	87.	81.74	0.0075	450	450	6.0	1.080	58	5	2	6.20	83.59	-2	0
18	0 *														
7	15	25.	81.09	0.0139	600	600	6.0	0.000	10	5	2	7.00	84.14	-2	0
7	16	31.	80.74	0.0037	450	450	6.0	0.000	10	5	2	7.00	83.64	-2	0
7	17	36.	80.63	0.0032	450	450	6.0	0.000	10	5	2	7.00	83.23	-2	0
7	18	45.	80.51	0.0026	450	450	6.0	0.000	10	5	2	7.00	83.06	-2	0
7	19	83.	80.40	0.0014	450	450	6.0	0.000	10	5	2	7.00	83.18	-2	0
7	20	77.	80.28	0.0015	450	450	6.0	0.000	10	5	2	7.00	82.96	-2	0
1	11	78.	80.16	0.0260	600	600	6.0	0.000	10	5	2	13.10	83.34	-2	0
1	12	46.	78.14	0.0440	600	600	6.0	0.000	10	5	2	13.10	82.11	-2	0
19	0	20.	79.62	0.0219	150	150	6.0	0.000	10	5	2	0.00	84.86	-2	0
19	1	38.	79.18	0.0115	150	150	6.0	0.000	10	5	2	0.00	84.33	-2	0
19	2	7.	78.74	0.0627	150	150	6.0	0.000	10	5	2	0.10	83.75	-2	0
19	3	32.	78.31	0.0137	225	225	6.0	0.000	10	5	2	0.20	83.11	-2	0
19	4	48.	77.87	0.0091	225	225	6.0	0.000	10	5	2	0.30	82.68	-2	0
19	5	14.	77.43	0.0313	225	225	6.0	0.000	10	5	2	0.40	82.54	-2	0
19	6	32.	76.99	0.0137	225	225	6.0	0.000	10	5	2	0.50	82.02	-2	0
19	7	23.	76.55	0.0191	225	225	6.0	0.000	10	5	2	0.60	81.74	-2	0
1	13	60.	76.11	0.0302	600	600	6.0	0.000	10	5	2	13.70	81.53	-2	0
1	14	98.	74.30	0.0185	600	600	6.0	0.000	10	5	2	13.70	77.98	-2	0
20	0	58.	77.32	0.0417	225	225	6.0	0.000	10	5	2	0.20	80.63	-2	0
20	1	66.	74.91	0.0366	225	225	6.0	0.000	10	5	2	0.40	77.54	-2	0
1	15	125.	72.49	0.0093	600	600	6.0	0.000	10	5	2	14.10	74.44	-2	0
21	0	35.	83.22	0.0129	150	150	6.0	0.000	10	5	2	0.00	84.98	-2	0
21	1	51.	82.77	0.0089	150	150	6.0	0.000	10	5	2	0.10	83.93	-2	0
21	2	7.	82.32	0.1796	150	150	6.0	0.000	10	5	2	0.20	82.87	-2	0
21	3	55.	81.06	0.0229	150	150	6.0	0.000	10	5	2	0.30	81.82	-2	0
21	4	38.	79.80	0.0152	225	225	6.0	0.000	10	5	2	0.40	80.76	-2	0
21	5	10.	79.22	0.0183	225	225	6.0	0.000	10	5	2	0.50	79.71	-2	0

													A(xiv)		
21	6	43.	79.04	0.0371	225	225	6.0	0.000	10	5	2	0.60	78.65	-2	0
21	7	59.	77.45	0.0522	300	300	6.0	0.000	10	5	2	0.70	77.60	-2	0
21	8	53.	74.37	0.0573	300	300	6.0	0.000	10	5	2	0.80	76.54	-2	0
1	16	66.	71.33	0.0174	600	600	6.0	0.000	10	5	2	14.90	73.68	-2	0
22	0	100.	103.25	0.0827	150	150	6.0	0.000	10	5	2	0.20	104.89	-2	0
22	1	100.	94.98	0.0827	150	150	6.0	0.000	10	5	2	0.40	96.92	-2	0
22	2	100.	86.72	0.0827	150	150	6.0	0.000	10	5	2	0.70	88.95	-2	0
22	3	100.	78.45	0.0827	150	150	6.0	0.000	10	5	2	1.00	80.97	-2	0
1	17	96.	70.18	0.0064	600	600	6.0	0.000	10	5	2	15.90	73.00	-2	0
1	18	102.	69.57	0.0080	600	600	6.0	0.000	10	5	2	15.90	72.24	-2	0
1	19	188.	68.76	0.0059	600	600	6.0	0.000	10	5	2	15.90	71.70	-2	0
23	0	671.	82.98	0.0229	450	450	6.0	0.000	10	5	2	0.20	84.58	-2	5
24	0 *														
1	20	124.	67.65	0.0110	600	600	6.0	0.000	10	5	2	20.10	70.27	-2	0
1	21	117.	66.28	0.0131	600	600	6.0	0.000	10	5	2	20.10	69.20	-2	0
25	0	450.	87.91	0.0515	375	375	6.0	0.000	10	5	2	0.80	89.50	-2	3
1	22	52.	64.75	0.0124	600	600	6.0	0.000	10	5	2	20.90	68.33	-2	0
1	23	94.	64.10	0.0352	600	600	6.0	0.000	10	5	2	20.90	69.86	-2	0
700	1 %%														
500	0	96.	60.64	0.0155	450	450	6.0	0.000	10	5	2	20.90	69.37	-2	0
26	0	601.	106.30	0.0524	450	450	6.0	0.000	10	5	2	0.40	107.27	-2	4
26	1	72.	74.83	0.0468	450	450	6.0	0.000	10	5	2	0.40	77.27	-2	1
27	0	657.	105.77	0.0522	450	450	6.0	0.000	10	5	2	0.40	109.15	-2	4
26	2	277.	71.46	0.0062	450	450	6.0	0.000	10	5	2	0.80	74.31	-2	4
28	0	168.	75.14	0.0322	375	375	6.0	0.000	10	5	2	0.40	76.88	-2	3
26	3	80.	69.74	0.1322	475	475	6.0	0.000	10	5	2	1.20	71.70	-2	2
500	1	47.	59.16	0.0104	450	450	6.0	0.000	10	5	2	22.10	67.59	-2	0
500	2	104.	58.67	0.0091	450	450	6.0	0.000	10	5	2	22.10	68.41	-2	0
29	0 *														
500	3	87.	57.73	0.0070	450	450	6.0	0.000	10	5	2	23.30	65.05	-2	0
30	0 *														
500	4	105.	57.11	0.0072	450	450	6.0	0.000	10	5	2	23.50	64.32	-2	0
31	0 *														
500	5	160.	56.36	0.0037	450	450	6.0	0.000	10	5	2	26.60	63.13	-2	0
800	1 $$														
600	0 ##	94.	59.79	0.0140	800	800	6.0	0.000	10	5	2	0.01	69.37	-2	0
600	1	83.	58.47	0.0101	850	850	6.0	0.000	10	5	2	0.02	67.61	-2	0
600	2	66.	57.64	0.0114	850	850	6.0	0.000	10	5	2	0.02	68.11	-2	0
600	3	88.	56.89	0.0087	900	900	6.0	0.000	10	5	2	0.02	64.95	-2	0
600	4	108.	56.12	0.0110	900	900	6.0	0.000	10	5	2	0.02	64.42	-2	0
600	5	149.	54.93	0.0067	975	975	6.0	0.000	10	5	2	0.02	62.98	-2	0
32	0 *														
700	2 %%														
500	6	102.	55.59	0.0091	450	450	6.0	0.000	10	5	2	27.40	61.34	-2	0
500	7	131.	54.66	0.0136	375	375	6.0	0.000	10	5	2	27.40	59.65	-2	0
800	2 $$														
600	6 ##	99.	53.94	0.0059	975	975	6.0	0.000	10	5	2	0.02	61.62	-2	0
600	7	80.	53.35	0.0249	850	850	6.0	0.000	10	5	2	0.02	59.10	-2	0
600	8	62.	51.36	0.0168	825	825	6.0	0.000	10	5	2	0.02	56.59	-2	0
700	3 %%														
500	8	7.	52.67	0.1040	375	375	6.0	0.000	10	5	2	27.40	58.76	-2	0
33	0 *														
500	9	78.	51.94	0.0307	375	375	6.0	0.000	10	5	2	28.20	59.12	-2	0
500	10	108.	49.54	0.0073	375	375	6.0	0.000	10	5	2	28.20	54.22	-2	0
500	11	98.	48.76	0.0080	375	375	6.0	0.000	10	5	2	28.20	51.38	-2	0
34	0	264.	60.54	0.0476	225	225	6.0	0.000	10	5	2	0.80	63.87	-2	3
35	0	177.	59.89	0.0673	150	150	6.0	0.000	10	5	2	0.80	61.75	-2	3
36	0 *														
500	12	190.	47.97	0.0095	375	375	6.0	0.000	10	5	2	35.80	50.10	-2	0
37	0 *														
500	13	56.	46.18	0.0047	450	450	6.0	0.000	10	5	2	35.80	47.93	-2	0
800	3 $$														
600	9 ##	117.	50.31	0.0197	1050	1050	6.0	0.000	10	5	2	0.02	59.01	-2	0
600	10	111.	48.01	0.0070	1050	1050	6.0	0.000	10	5	2	0.02	51.74	-2	0
600	11	111.	47.23	0.0074	1050	1050	6.0	0.000	10	5	2	0.02	51.51	-2	0
600	12	127.	46.41	0.0073	1050	1050	6.0	0.000	10	5	2	0.02	49.96	-2	0
600	13	98.	45.48	0.0104	1050	1050	6.0	0.000	10	5	2	0.02	49.88	-2	0
38	0	68.	53.49	0.0123	375	375	6.0	0.000	10	5	2	0.00	54.88	-2	0
39	0 *														
38	1	88.	52.66	0.0130	375	375	6.0	0.000	10	5	2	1.80	55.81	-2	0
38	2	90.	51.51	0.0127	375	375	6.0	0.000	10	5	2	1.80	53.73	-2	0
40	0 *														
41	0 *														
38	3	91.	50.37	0.0105	375	375	6.0	0.000	10	5	2	5.60	51.21	-2	0
38	4	88.	49.42	0.0101	375	375	6.0	0.000	10	5	2	5.60	50.72	-2	0
42	0 *														
38	5	81.	48.53	0.0105	375	375	6.0	0.000	10	5	2	7.40	50.23	-2	0
38	6	117.	47.67	0.0107	375	375	6.0	0.000	10	5	2	7.40	49.12	-2	0
38	7	13.	46.42	0.0389	300	300	6.0	0.000	10	5	2	7.40	48.20	-2	0
700	4 %%														
500	14	88.	45.88	0.0091	450	450	6.0	0.000	10	5	2	43.20	48.22	-2	0
500	15	56.	45.07	0.0108	375	375	6.0	0.000	10	5	2	43.20	47.92	-2	0
43	0 *														
43	1	93.	67.12	0.0080	450	450	6.0	0.000	10	5	2	0.00	69.23	-2	0
43	2	190.	66.37	0.0076	450	450	6.0	0.000	10	5	2	0.00	68.62	-2	0
44	0	246.	67.81	0.0117	225	225	6.0	0.000	10	5	2	0.00	69.00	-2	4
43	3	43.	64.92	0.0089	450	450	6.0	0.000	10	5	2	0.00	68.96	-2	0

													A(xiv)		
43	4	50.	64.54	0.0200	450	450	6.0	0.000	10	5	2	0.00	67.71	-2	0
43	5	80.	63.54	0.0168	450	450	6.0	0.000	10	5	2	0.00	66.78	-2	0
45	0 *														
43	6	68.	62.19	0.0152	450	450	6.0	0.000	10	5	2	0.00	65.07	-2	0
46	0	843.	74.81	0.0162	225	225	6.0	0.000	10	5	2	0.00	76.39	-2	5
43	7	56.	61.16	0.0029	450	450	6.0	0.000	10	5	2	0.00	64.19	-2	0
47	0	619.	71.32	0.0167	150	150	6.0	0.000	10	5	2	0.00	73.58	-2	5
48	0 *														
43	8	23.	60.99	0.0198	450	450	6.0	0.000	10	5	2	0.00	63.00	-2	0
43	9	37.	60.54	0.0140	450	450	6.0	0.000	10	5	2	0.00	62.68	-2	0
49	0	215.	67.99	0.0371	150	150	6.0	0.000	10	5	2	0.00	69.74	-2	3
43	10	53.	60.02	0.0115	838	838	6.0	0.000	10	5	2	0.00	62.30	-2	0
43	11	104.	59.41	0.0083	838	838	6.0	0.000	10	5	2	0.00	61.89	-2	0
43	12	55.	58.55	0.0200	838	838	6.0	0.000	10	5	2	0.00	61.28	-2	0
50	0 *														
51	0	509.	68.20	0.0211	309	309	6.0	0.000	10	5	2	0.00	69.56	-2	5
43	13	37.	57.45	0.0221	900	900	6.0	0.000	10	5	2	0.00	60.76	-2	0
43	14	100.	56.63	0.0082	900	900	6.0	0.000	10	5	2	0.00	58.98	-2	0
52	0	444.	66.71	0.0246	300	300	6.0	0.000	10	5	2	0.00	67.97	-2	6
43	15	112.	55.81	0.0133	900	900	6.0	0.000	10	5	2	0.00	57.64	-2	0
53	0 *														
54	0 *														
43	16	147.	54.33	0.0171	900	900	6.0	0.000	10	5	2	0.00	55.87	-2	0
55	0 *														
43	17	83.	51.82	0.0097	900	900	6.0	0.000	10	5	2	0.00	54.88	-2	0
43	18	168.	51.01	0.0156	900	900	6.0	0.000	10	5	2	0.00	55.01	-2	0
43	19	88.	48.40	0.0067	900	900	6.0	0.000	10	5	2	0.00	51.21	-2	0
43	20	92.	47.81	0.0071	900	900	6.0	0.000	10	5	2	0.00	50.77	-2	0
43	21	95.	47.15	0.0175	900	900	6.0	0.000	10	5	2	0.00	50.05	-2	0
43	22	68.	45.49	0.0060	900	900	6.0	0.000	10	5	2	0.00	48.33	-2	0
56	0 *														
43	23	52.	45.08	0.0120	900	900	6.0	0.000	10	5	2	0.00	47.90	-2	0
800	4 $$														
600	14 ##	149.	44.46	0.0075	1140	1140	6.0	0.000	10	5	2	0.02	48.18	-2	0
57	0 *														
700	5 %%														
500	16	94.	44.33	0.0061	375	375	6.0	0.000	10	5	2	44.20	46.25	-2	0
500	17	113.	43.75	0.0071	450	450	6.0	0.000	10	5	2	44.20	46.94	-2	0
58	0 *														
500	18	58.	42.95	0.0071	600	600	6.0	0.000	10	5	2	45.20	48.04	-2	0
500	19	97.	42.54	0.0068	600	600	6.0	0.000	10	5	2	45.20	47.86	-2	0
59	0 *														
60	0 *														
61	0 *														
59	1	58.	86.95	0.0662	450	450	6.0	0.000	10	5	2	44.00	89.75	-2	0
62	0 *														
59	2	66.	83.12	0.0017	450	450	6.0	0.000	10	5	2	46.00	85.26	-2	0
63	0 *														
64	0 *														
59	3	192.	83.00	0.0046	450	450	6.0	0.000	10	5	2	51.00	86.30	-2	1
65	0 *														
66	0 *														
59	4	815.	82.11	0.0494	450	450	6.0	0.000	10	5	2	53.00	84.61	-2	5
67	0 *														
500	20	88.	41.88	0.0090	750	750	6.0	0.000	10	5	2	99.20	47.62	-2	0
68	0 *														
500	21	30.	41.08	0.0130	600	600	6.0	0.000	10	5	2	103.20	45.40	-2	0
800	5 $$														
600	15 ##	98.	43.34	0.0046	1140	1140	6.0	0.000	10	5	2	0.02	46.03	-2	0
600	16	90.	42.89	0.0061	1140	1140	6.0	0.000	10	5	2	0.02	46.95	-2	0
69	0 *														
600	17	70.	42.33	0.0130	1140	1140	6.0	0.000	10	5	2	0.02	47.57	-2	0
600	18	87.	41.42	0.0105	1140	1140	6.0	0.000	10	5	2	0.02	47.86	-2	0
600	19	78.	40.51	0.0117	1080	1080	6.0	0.000	10	5	2	0.02	46.12	-2	0
70	0 *														
600	20	38.	39.60	0.0114	1200	1200	6.0	0.000	10	5	2	5.02	45.40	-2	0

Total catchment area = 312.410 Ha

* INDICATES SEWERED SUBAREA
** INDICATES STORM OVERFLOW
%% INDICATES CROSS CONNECTION O/F MANHOLE
$$ INDICATES CROSS CONNECTION PIPE
INDICATES C.C. PIPE JOINED AT U/S NODE
++ INDICATES ON-LINE STORAGE TANK

****

CROSS CONNECTION OVERFLOW DETAILS
===== ========== ======== =======

THERE ARE 5 CROSS CONNECTIONS

OVERFLOW DETAILS

OVERFLOW WEIR NO	OVERFLOW WEIR REF	D/S C.C. PIPE NO	D/S C.C. PIPE REF	D/S M/H NO DIRECT	D/S M/H NO OVER	OVERFLOW TYPE	WEIR LENGTH (m)	TOP GL (m)	TOP INV LEV (m)	WEIR DEPTH (mm)	FLOW SETTING (lit/s)	COEF OF DISCHARGE
700	1	800	1	90	106	0	2.0	69.37	60.789	350	0	0.000
700	2	800	2	114	117	0	1.2	61.34	55.770	350	0	0.000
700	3	800	3	121	133	1	2.0	58.76	52.880	150	0	0.000
700	4	800	4	151	191	0	1.5	48.22	45.915	375	0	0.000
700	5	800	5	194	217	1	2.5	46.25	44.472	175	0	0.000

BRIDGING PIPE DETAILS

PIPE NO	PIPE REF.	D/STREAM BN NO	D/STREAM PIPE NO	PIPE LENGTH (m)	INLET INV LEV (m)	OUTLET INV LEV (m)	DIAMETER (mm)	SLOPE
800	1	600	0	3.	60.643	59.790	700	0.2843
800	2	600	6	2.	55.590	55.350	450	0.1200
800	3	600	9	4.	51.215	50.920	450	0.0738
800	4	600	14	3.	45.875	44.860	450	0.3383
800	5	600	15	3.	43.570	43.440	450	0.0433

****

THERE ARE NO STORAGE TANKS

THERE ARE NO STORM OVERFLOWS

TERMINATION PIPE

THERE ARE 2 DUMMY OUTFALL PIPE/S

BRANCH NO	PIPE NO	END INVERT LEVEL
500	22	40.690
600	21	39.170

****

SEWERED SUB-AREA DETAILS

There are 50 sewered sub-areas
SYSTEM NAME BOTH ELL T CA CHME T TO AL I FLOW -- P RALL L PI E VE SION

BN NO	PIPE NO	MAIN LEN	MAIN SLOPE	OUTFALL DIA	TOT AREA	% IMP	% ROOF	SLOPE INDEX	DWF l/s
1	0	325.	0.0243	300	6.520	40	5	2	4.000
2	0	311.	0.0435	225	3.780	40	5	2	0.300
5	0	497.	0.0182	375	4.430	27	5	1	0.800
6	0	534.	0.0046	300	2.310	14	11	1	0.600
7	0	1010.	0.0340	375	29.088	27	13	3	2.000
8	0	52.	0.0220	300	2.060	2	21	1	0.600
9	0	47.	0.0292	300	0.490	31	5	1	0.200
10	0	52.	0.0343	225	0.310	11	5	1	0.200
11	0	292.	0.0878	150	3.730	28	5	2	0.800
12	0	230.	0.0824	150	2.160	28	5	2	0.600
13	0	90.	0.0183	150	0.590	32	5	1	0.100
14	0	215.	0.0469	150	1.470	26	5	2	0.200
15	0	94.	0.0165	200	0.690	30	5	1	0.100
16	0	100.	0.0160	150	0.880	3	10	1	0.100
17	0	310.	0.0383	225	3.830	20	10	2	0.800
18	0	175.	0.0470	225	3.730	58	5	1	0.800
24	0	667.	0.0308	300	9.540	58	5	3	4.000
29	0	675.	0.0510	375	12.168	58	5	3	1.200
30	0	215.	0.0751	225	1.290	24	5	3	0.200
31	0	426.	0.0862	300	9.369	58	5	3	3.100
32	0	314.	0.0544	300	3.760	24	5	3	0.800
33	0	246.	0.0875	300	2.970	24	5	3	0.800
36	0	627.	0.0468	300	6.300	40	5	3	6.000
37	0	212.	0.0321	225	1.459	18	5	1	0.000
39	0	206.	0.0560	150	2.358	20	5	3	1.800
40	0	276.	0.0444	300	3.438	20	5	2	1.900
41	0	292.	0.0249	225	3.242	20	5	1	1.900
42	0	163.	0.0193	150	3.045	15	5	1	1.800
43	0	680.	0.0298	450	7.171	30	10	2	0.000
45	0	182.	0.0786	450	5.206	20	8	2	0.000
48	0	569.	0.0464	375	13.851	30	10	3	0.000
50	0	585.	0.0597	300	11.198	30	10	3	0.000

							A(xiv)
1	17	0.398	4.20	0.5615	0.1693	0.2692	
1	18	0.443	4.44	0.6553	0.1390	0.2057	
1	19	0.382	7.92	0.6606	0.1603	0.1791	
23	0	0.349	132.22	0.2732	0.3503	0.3765	
1	20	0.522	5.33	0.7768	0.0615	0.1617	
1	21	0.570	5.05	0.8117	0.1092	0.0790	
25	0	0.321	233.11	0.1228	0.4323	0.4449	
1	22	0.554	2.42	0.3307	0.4686	0.2007	
1	23	1.508	1.61	0.2343	0.4233	0.3424	
500	0	0.287	19.07	0.8567	0.1268	0.0165	
26	0	0.528	117.95	0.2884	0.4935	0.2181	
26	1	0.499	15.54	0.1240	0.4576	0.4184	
27	0	0.527	128.47	0.2892	0.5269	0.1839	
26	2	0.182	57.26	0.2162	0.4402	0.3436	
28	0	0.254	92.31	0.2596	0.3764	0.3640	
26	3	0.970	13.68	0.2893	0.1101	0.6006	
500	1	0.235	9.86	0.2051	0.4089	0.3860	
500	2	0.220	20.60	0.7492	0.0265	0.2244	
500	3	0.193	17.40	0.6447	0.1054	0.2499	
500	4	0.195	20.80	0.6801	0.0120	0.3080	
500	5	0.140	31.22	0.5269	0.2518	0.2213	
800	1	4.007	0.10	0.3958	0.2905	0.3137	
600	0	1.270	0.92	0.4836	0.3197	0.1967	
600	1	1.262	0.60	0.3513	0.3669	0.2818	
600	2	1.342	0.50	0.3889	0.4685	0.1426	
600	3	1.366	0.47	0.5894	0.3550	0.0556	
600	4	1.540	0.57	0.7032	0.2735	0.0233	
600	5	1.478	0.50	0.5430	0.1414	0.3156	
500	6	0.220	20.22	0.7468	0.0362	0.2170	
500	7	0.165	67.65	0.6484	0.1125	0.2391	
800	2	0.799	0.68	0.3867	0.2869	0.3264	
600	6	1.396	0.35	0.4079	0.3401	0.2519	
600	7	1.988	0.58	0.1565	0.4816	0.3619	
600	8	1.509	0.55	0.2944	0.4192	0.2864	
500	8	0.456	5.58	0.4242	0.2455	0.3303	
500	9	0.248	41.07	0.4398	0.2887	0.2715	
500	10	0.121	56.22	0.3775	0.2728	0.3497	
500	11	0.126	51.19	0.4362	0.1207	0.4431	
34	0	0.078	2131.62	0.2383	0.3742	0.3875	
35	0	0.031	12825.14	0.2115	0.4508	0.3377	
500	12	0.137	97.25	0.1937	0.3282	0.4781	
500	13	0.157	11.57	0.3784	0.3311	0.2905	
800	3	0.627	1.05	0.3729	0.2672	0.3599	
600	9	3.102	0.27	0.3570	0.4513	0.1917	
600	10	1.846	0.26	0.4929	0.3064	0.2007	
600	11	1.900	0.26	0.5200	0.3047	0.1753	
600	12	1.888	0.29	0.5374	0.2428	0.2198	
600	13	2.253	0.24	0.4466	0.3649	0.1885	
38	0	0.157	36.13	0.4985	0.1982	0.3033	
38	1	0.161	46.13	0.8367	0.0793	0.0840	
38	2	0.159	47.13	0.8344	0.0652	0.1004	
38	3	0.144	47.66	0.7944	0.0365	0.1691	
38	4	0.142	46.16	0.7840	0.0502	0.1658	
38	5	0.145	42.65	0.7842	0.0950	0.1208	
38	6	0.146	60.67	0.2241	0.4864	0.2895	
38	7	0.153	26.66	0.3099	0.3234	0.3667	
500	14	0.220	17.58	0.7294	0.1097	0.1610	
500	15	0.147	30.13	0.3944	0.2750	0.3306	
43	1	0.206	18.53	0.6978	0.0755	0.2267	
43	2	0.201	36.81	0.2861	0.2759	0.4380	
44	0	0.039	2012.85	0.2076	0.5633	0.2291	
43	3	0.217	9.11	0.4402	0.2355	0.3243	
43	4	0.326	10.41	0.2105	0.4603	0.3292	
43	5	0.299	16.06	0.3602	0.2280	0.4118	
43	6	0.284	13.80	0.5901	0.2888	0.1211	
46	0	0.045	6712.10	0.3836	0.3345	0.2819	
43	7	0.125	11.59	0.1102	0.3970	0.4928	
47	0	0.015	44353.06	0.3228	0.3314	0.3458	
43	8	0.324	5.33	0.4439	0.2082	0.3479	
43	9	0.272	7.97	0.1630	0.5293	0.3077	
49	0	0.023	15526.83	0.3852	0.4579	0.1569	
43	10	1.299	0.44	0.2443	0.5466	0.2091	
43	11	1.104	0.79	0.6079	0.2537	0.1383	
43	12	1.715	0.46	0.3284	0.4836	0.1880	
51	0	0.122	747.86	0.1617	0.6563	0.1820	
43	13	2.180	0.24	0.2335	0.5203	0.2462	
43	14	1.325	0.53	0.5815	0.2970	0.1215	
52	0	0.122	772.60	0.2695	0.4033	0.3272	
43	15	1.689	0.58	0.2618	0.2726	0.4656	
43	16	1.915	0.75	0.4111	0.3745	0.2144	
43	17	1.440	0.45	0.3256	0.3835	0.2909	
43	18	1.829	0.84	0.8317	0.0962	0.0721	
43	19	1.204	0.47	0.4727	0.3569	0.1704	
43	20	1.233	0.49	0.5039	0.3358	0.1603	
43	21	1.939	0.51	0.4252	0.3785	0.1963	
43	22	1.139	0.38	0.3705	0.4687	0.1608	
43	23	1.607	0.31	0.1680	0.5738	0.2582	

53	0	374.	0.0802	375	8.644	30	10	3	0.000
54	0	206.	0.0461	300	3.210	25	5	2	0.000
55	0	499.	0.0669	450	5.599	25	10	3	0.000
56	0	769.	0.0206	300	16.245	20	5	2	0.000
57	0	650.	0.0147	300	8.153	16	32	1	1.000
58	0	383.	0.0447	370	3.113	36	5	2	1.000
59	0	405.	0.0660	225	17.801	58	5	3	18.000
60	0	754.	0.0154	375	19.455	40	12	2	18.000
61	0	542.	0.0226	225	3.536	24	5	2	8.000
62	0	479.	0.0113	300	3.599	14	5	1	2.000
63	0	111.	0.0217	225	0.982	48	5	1	2.000
64	0	483.	0.0305	225	5.010	32	5	2	3.000
65	0	951.	0.0032	300	10.334	58	5	2	1.000
66	0	191.	0.0270	225	2.335	24	5	2	1.000
67	0	406.	0.0364	600	6.226	46	5	2	1.000
68	0	695.	0.0299	300	7.145	39	20	2	4.000
69	0	383.	0.0447	900	3.113	36	5	2	0.000
70	0	889.	0.0704	500	23.477	40	30	3	5.000

****

HYDRAULIC DETAILS OF PIPES

PIPE NO	BRANCH NO	QFB M**3/S	DARCY BETA	MUSKINGUM C1	C2	C3
1	1	0.128	25.57	0.3848	0.1106	0.5046
1	2	0.218	5.72	0.5958	0.1803	0.2239
1	3	0.103	5.74	0.3124	0.1057	0.5819
1	4	0.252	4.77	0.3006	0.2939	0.4055
3	0	0.092	1197.90	0.2875	0.2340	0.4785
1	5	0.102	18.04	0.0184	0.2864	0.6951
1	6	0.142	11.77	0.2671	0.3468	0.3861
1	7	0.197	2.89	0.4328	0.3044	0.2628
4	0	0.073	2154.18	0.3567	0.2141	0.4292
1	8	0.134	16.50	0.3010	0.1962	0.5028
1	9	0.490	41.78	0.5415	0.1660	0.2925
6	1	0.274	12.08	0.4219	0.3702	0.2079
1	10	0.238	48.57	0.6841	0.1892	0.1267
7	1	0.180	117.15	0.2244	0.3512	0.4244
7	2	0.191	39.60	0.4962	0.2103	0.2935
7	3	0.115	154.50	0.4189	0.1782	0.4029
7	4	0.148	154.38	0.6767	0.1820	0.1413
7	5	0.304	38.22	0.3492	0.2567	0.3491
7	6	0.078	44.82	0.3111	0.1240	0.5649
7	7	0.239	6.59	0.2032	0.3769	0.4199
7	8	0.222	12.87	0.6968	0.2716	0.0315
7	9	0.221	15.13	0.7145	0.1887	0.0968
7	10	0.172	9.87	0.4384	0.3957	0.1660
7	11	0.210	16.83	0.6942	0.1272	0.1785
7	12	0.107	15.77	0.0294	0.3223	0.6482
7	13	0.379	5.14	0.1966	0.4473	0.3561
7	14	0.200	17.40	0.6679	0.1065	0.2256
7	15	0.946	0.54	0.3532	0.2446	0.4022
7	16	0.141	6.86	0.1254	0.5848	0.2899
7	17	0.130	7.81	0.0581	0.5528	0.3890
7	18	0.117	9.52	0.3316	0.2102	0.4582
7	19	0.086	16.75	0.1950	0.4279	0.3771
7	20	0.089	15.60	0.1892	0.4354	0.3754
1	11	0.802	3.47	0.2925	0.4017	0.3058
1	12	1.045	2.17	0.2464	0.4656	0.2880
19	0	0.018	1497.58	0.2473	0.3207	0.4320
19	1	0.013	2778.14	0.8408	0.1396	0.0196
19	2	0.030	575.97	0.2763	0.3721	0.3516
19	3	0.042	267.35	0.4025	0.3842	0.2133
19	4	0.034	393.46	0.7637	0.1487	0.0876
19	5	0.063	125.82	0.3314	0.3529	0.3157
19	6	0.042	267.35	0.4023	0.3843	0.2134
19	7	0.049	196.54	0.3770	0.4568	0.1662
1	13	0.865	2.74	0.3282	0.2282	0.4436
1	14	0.677	4.28	0.3645	0.2437	0.3918
20	0	0.073	470.36	0.3908	0.3217	0.2875
20	1	0.068	533.13	0.4689	0.2348	0.2963
1	15	0.479	5.37	0.7324	0.0445	0.2231
21	0	0.014	2564.10	0.4506	0.1967	0.3527
21	1	0.011	3704.86	0.4343	0.3397	0.2260
21	2	0.050	575.24	0.2616	0.3217	0.4167
21	3	0.018	3975.06	0.3406	0.1800	0.4794
21	4	0.044	314.37	0.4935	0.3243	0.1822
21	5	0.048	94.57	0.2514	0.4766	0.2720
21	6	0.069	352.95	0.1897	0.4270	0.3833
21	7	0.177	102.91	0.1389	0.4452	0.4159
21	8	0.186	92.96	0.2735	0.5040	0.2225
1	16	0.656	2.98	0.3464	0.4103	0.2433
22	0	0.034	7140.62	0.2910	0.3791	0.3299
22	1	0.034	7140.62	0.2509	0.4791	0.2700
22	2	0.034	7140.62	0.2810	0.4421	0.2769
22	3	0.034	7140.62	0.3690	0.2766	0.3544

						A(xiv)
800	4	1.343	0.87	0.3264	0.3882	0.2854
600	14	2.375	0.22	0.5557	0.1968	0.2475
500	16	0.110	49.22	0.6443	0.1044	0.2513
500	17	0.194	22.30	0.3387	0.3249	0.3364
500	18	0.419	2.66	0.5244	0.3978	0.0778
500	19	0.411	4.24	0.5907	0.1628	0.2465
59	1	0.593	11.90	0.2742	0.5938	0.1320
59	2	0.095	13.51	0.3397	0.4568	0.2035
59	3	0.157	38.24	0.6537	0.2645	0.0818
59	4	0.513	159.17	0.4731	0.2086	0.3183
500	20	0.858	1.21	0.6402	0.2942	0.0656
500	21	0.915	0.61	0.3397	0.3227	0.3376
800	5	0.480	0.87	0.3224	0.2684	0.4092
600	15	1.868	0.15	0.2189	0.4251	0.3560
600	16	2.151	0.14	0.3664	0.4319	0.2017
600	17	3.132	0.12	0.2430	0.5357	0.2213
600	18	2.810	0.14	0.3213	0.4406	0.2381
600	19	2.568	0.17	0.3913	0.4736	0.1351
600	20	3.354	0.06	0.3817	0.3217	0.2966

****

HYDRAULIC DETAILS OF SEWERED SUB AREAS

PIPE NO	BRANCH NO	EQUIV DIA	EQUIV SLOPE	NO OF EQUIV PIPES
1	0	0.317	0.0182	3
2	0	0.238	0.0326	3
5	0	0.397	0.0137	4
6	0	0.317	0.0035	9
7	0	0.397	0.0255	6
8	0	0.317	0.0165	1
9	0	0.317	0.0219	1
10	0	0.238	0.0257	1
11	0	0.158	0.0659	2
12	0	0.158	0.0618	2
13	0	0.158	0.0137	2
14	0	0.158	0.0352	2
15	0	0.211	0.0124	2
16	0	0.158	0.0120	2
17	0	0.238	0.0287	3
18	0	0.238	0.0353	2
24	0	0.317	0.0231	5
29	0	0.397	0.0382	3
30	0	0.238	0.0563	2
31	0	0.317	0.0646	2
32	0	0.317	0.0408	2
33	0	0.317	0.0656	1
36	0	0.317	0.0351	4
37	0	0.238	0.0241	2
39	0	0.158	0.0420	2
40	0	0.317	0.0333	2
41	0	0.238	0.0187	3
42	0	0.158	0.0145	3
43	0	0.476	0.0224	4
45	0	0.476	0.0589	1
48	0	0.397	0.0348	3
50	0	0.317	0.0448	3
53	0	0.397	0.0602	2
54	0	0.317	0.0346	2
55	0	0.476	0.0502	2
56	0	0.317	0.0155	7
57	0	0.317	0.0110	7
58	0	0.391	0.0335	2
59	0	0.238	0.0495	3
60	0	0.397	0.0115	6
61	0	0.238	0.0170	5
62	0	0.317	0.0085	6
63	0	0.238	0.0163	2
64	0	0.238	0.0229	4
65	0	0.317	0.0024	19
66	0	0.238	0.0203	2
67	0	0.635	0.0273	2
68	0	0.317	0.0224	5
69	0	0.953	0.0335	1
70	0	0.529	0.0528	3

****

****

APPENDIX B

COMPUTER LISTING OF THE GRAPHICAL PLOTTING PROGRAM

[PROGRAM GPLOT.FOR]

APPENDIX B
COMPUTER LISTING OF GRAPHICAL PLOTTING PROGRAM

```
C      ***************************************************
C      *                                                 *
C      * GRAPHICAL PROGRAM TO PLOT RAINFALL HYETOGRAPHS &  *
C      *  RUNOFF HYDROGRAPHS FOR A REAL EVENT AND/OR FOR A *
C      *                    DESIGN STORM                 *
C      *  --- by W.K. AU-YEUNG & OLIVER AU-YEUNG          *
C      *                                                 *
C      ***************************************************
C
C    FURTHER UPDATE ON 31/5/88 -- INCL DWF SETTING ROUTINES
C    LAST UPDATE ON 17/5/88
C    NEW DEVELOPED PROGRAM FOR READING VAXPLOT.DAT FILE SEND FROM
C    APRICOT WITHOUT DELETING THE FIRST 2 COLUMNS
C
C     MAIN CHANGES INCLUDED -- (1) READ IN THE TIME & FLOWRATE COLUMNS
C                              (2) CALCULATE & FILL UP INTERVAL GAPS
C                              (3) INFORMATION SUBROUTINE
C
      REAL X(2000),Y(2000),YV(2000),YY(2000),YD(2000),YH(2000)
      REAL TM(2000),TE(2000),YT(2000),SP(2000)
      INTEGER IA(2000)
      CHARACTER*12 FILE1,FILE2
      CHARACTER SITE*12,RSITE*16,DATE*12,XDWF*12
      CHARACTER*2 CHOICE,SORP,GRID,INFO,SETT,ADDGRA
      REAL MAXF,DURA,MAXIN,INTER,SUM,ERF,STDU,SUN,RDATA
      REAL XLEN,YLEN,XHS,DNT,YOLD,YNEW,AA,DWF,TOT,DIFF
      INTEGER FILENO,DUMMY,DATANO,HYETO,RNINDO,RNIN,DATPT,FORM
C
      WRITE(5,65)
      WRITE(5,10)
      WRITE(5,15)
      WRITE(5,20)
      WRITE(5,25)
      WRITE(5,30)
      WRITE(5,35)
      WRITE(5,40)
      WRITE(5,45)
      WRITE(5,50)
      WRITE(5,55)
      WRITE(5,60)
C
      WRITE(5,11)
      READ(5,300)INFO
         IF(INFO.NE.'Y')GOTO 13
            CALL NEWS
13    CONTINUE
      WRITE(5,200)
      WRITE(5,350)
      READ(5,170)DATE
      WRITE(5,500)
      READ(5,520)XLEN
      WRITE(5,501)
      READ(5,520)YLEN
      WRITE(5,510)
      READ(5,300)GRID
      WRITE(5,12)
      READ(5,300)SETT
         IF(SETT.NE.'Y')GOTO 14
               WRITE(5,16)
               READ(5,130)DWF
               WRITE(5,17)
               READ(5,170)XDWF
14    CONTINUE
      WRITE(5,100)
      READ(5,110)FILENO
      WRITE(5,120)
      READ(5,130)DURA
      WRITE(5,140)
      READ(5,150)MAXF
      IDUT=IFIX(DURA)
      IMAX=IFIX(MAXF)
       WRITE(5,220)
       READ(5,230)HYETO
        WRITE(5,240)
        READ(5,130)STDU
         WRITE(5,250)
         READ(5,130)MAXIN
      CALL GINO
430   WRITE(5,450)
      READ(5,300)SORP
      IF(SORP.EQ.'S'.OR.SORP.EQ.'s')GOTO 460
```

```
      IF(SORP.EQ.'P'.OR.SORP.EQ.'p')GOTO 420
      IF(SORP.NE.'S'.OR.SORP.NE.'P')GOTO 430
460   CALL T4010
      GOTO 440
420   CALL CAL1044
440   CONTINUE
      CALL WINDOW(2)
      CALL PICCLE
C   POSITION, SCALE & DRAW THE X-AXIS
      IF(SORP.EQ.'P'.OR.SORP.EQ.'p')GOTO 618
      CALL AXIPOS(1,20.0,20.0,XLEN,1)
      GOTO 619
618   CALL AXIPOS(1,20.0,40.0,XLEN,1)
619   CALL AXISCA(1,10,0.0,DURA,1)
      CALL AXIDRA(-1,1,1)
C   POSITION, SCALE & DRAW THE Y-AXIS
      IF(SORP.EQ.'P'.OR.SORP.EQ.'p')GOTO 620
      CALL AXIPOS(1,20.0,20.0,YLEN,2)
      GOTO 621
620   CALL AXIPOS(1,20.0,40.0,YLEN,2)
621   CALL AXISCA(1,10,0.0,MAXF,2)
      CALL AXIDRA(1,-1,2)
C   SET UP THE GRID LINE FOR THE Y-AXIS
      IF(GRID.EQ.'Y'.OR.GRID.EQ.'y')GOTO 530
      GOTO 550
530   CONTINUE
        CALL BROKEN(5)
          DO 540 I=1,10
              CALL GRAMOV(0.5,(MAXF*REAL(I))/10.0)
              CALL GRALIN(DURA,(MAXF*REAL(I))/10.0)
540       CONTINUE
        CALL BROKEN(0)
550   CONTINUE
C
C   WRITE X-AXIS TITLE
      CALL HARCHA
           CALL CHASIZ(2.0,3.0)
              CALL GRAMOV(DURA*0.45,MAXF/-16.5)
      CALL CHAHOL(13HTIME (MINS)*.)
C
C   WRITE Y-AXIS TITLE
      CALL CHAANG(90.0)
         CALL GRAMOV(DURA/-20.0,MAXF*0.3)
      CALL CHAHOL(20HDISCHARGE (CUMECS)*.)
      CALL CHAANG(0.0)
C
C   WRITE TITLE, DURATION & VOLUME, ETC.
      CALL SOFCHA
         CALL CHASIZ(3.0,4.0)
            CALL ITALIC(30.0)
              CALL GRAMOV(DURA*0.55,MAXF*0.95)
            CALL CHAHOL(14HEVENT DATE: *.)
         CALL CHASTR(DATE)
      CALL ITALIC(0.0)
C
      CALL HARCHA
          CALL CHASIZ(2.0,3.0)
              CALL GRAMOV(DURA*0.55,MAXF*0.9)
          CALL CHAHOL(20HMAX. DURA (MINS): *.)
      CALL CHAFIX(DURA,7,2)
C
      CALL GRAMOV(DURA*0.55,MAXF*0.85)
      CALL CHAHOL(22HRUNOFF VOL.(CU.M.): *.)
C
      DO 1 I=1,FILENO
         CALL GRAMOV(0.0,0.0)
C
      ICOUNT=0
       WRITE(5,360)
         READ(5,300)ADDGRA
C
      IF(SORP.EQ.'S')GOTO 700
        WRITE(5,165)
         READ(5,170)FILE1
        WRITE(5,285)
        WRITE(5,290)
         READ(5,300)CHOICE
      IF(CHOICE.NE.'O')GOTO 84
        WRITE(5,83)
         READ(5,230)FORM
84     WRITE(5,195)
         READ(5,150)INTER
        WRITE(5,215)
         READ(5,170)SITE
      GOTO 710
700   CONTINUE
```

```
        WRITE(5,160)
         READ(5,170)FILE1
        WRITE(5,280)
         READ(5,300)CHOICE
      IF(CHOICE.NE.'O')GOTO 81
        WRITE(5,82)
         READ(5,230)FORM
81     WRITE(5,190)
         READ(5,150)INTER
        WRITE(5,210)
         READ(5,170)SITE
710   CONTINUE
      OPEN(UNIT=22,STATUS='OLD',READONLY,FILE=FILE1,
     1     ACCESS='SEQUENTIAL')
       DUMMY=I-(I/4)*4
          IF (DUMMY.EQ.1) THEN
                     CALL PENSEL(1,0.2,0)
            ELSEIF (DUMMY.EQ.2) THEN
                     CALL PENSEL(2,0.2,0)
              ELSEIF (DUMMY.EQ.3) THEN
                     CALL PENSEL(7,0.2,0)
             ELSE
                 CALL PENSEL(5,0.2,0)
          ENDIF
              READ(22,180)DATANO
              SUM=0.
              ERF=0.
C
           IF (CHOICE.EQ.'O') THEN
                     GOTO 666
             ELSEIF (CHOICE.EQ.'W') THEN
                     GOTO 777
              ELSE
                  GOTO 888
             ENDIF
C
777   CONTINUE
             READ(22,310)(Y(J),J=1,DATANO)
C             WRITE(5,320)(Y(J),J=1,DATANO)
      GOTO 999
888   CONTINUE
C             READ(22,330)(Y(J),J=1,DATANO)
              READ(22,*)(Y(J),J=1,DATANO)
               DO 663 J=1,DATANO
                 Y(J)=Y(J)/1000.
663             CONTINUE
C             WRITE(5,340)(Y(J),J=1,DATANO)
      GOTO 999
666   CONTINUE
           IF(FORM.EQ.1) THEN
                     GOTO 720
              ELSEIF(FORM.EQ.2) THEN
                     GOTO 730
              ELSE
                     GOTO 740
           ENDIF
C
740      READ(22,88)(TM(J),YT(J),J=1,DATANO)
C
      DO 750 IX=1,DATANO
         IM=IFIX(TM(IX))
         TE(IX)=FLOAT(IM)+((TM(IX)-FLOAT(IM))/0.6)
750   CONTINUE
C
      DO 760 IK=1,DATANO-1
        IF(TE(IK+1).EQ.0.0)TE(IK+1)=24.0
          AA=((TE(IK+1)-TE(IK))*60.0)/INTER
            SP(IK)=AA
          IA(IK)=NINT(AA)
        IF(TE(IK+1).EQ.24.0)TE(IK+1)=0.0
760   CONTINUE
C
      KA=1
      DO 770 KL=1,DATANO-1
        IF(KL.EQ.1)Y(1)=YT(1)
          IP=IA(KL)-1
        IF(IP.LT.1)GOTO 89
C
          YOLD=YT(KL)
          DNT=(YT(KL+1)-YT(KL))/SP(KL)
C
```

```
          DO 780 ITT=1,IP
            YOLD=YOLD+DNT
              KA=KA+1
            Y(KA)=YOLD
780       CONTINUE
89       CONTINUE
           KA=KA+1
           Y(KA)=YT(KL+1)
770      CONTINUE
C
         DATANO=KA
         WRITE(5,91)DATANO
         GOTO 555
C    -------------------------
730            READ(22,135)(Y(J),J=1,DATANO)
         GOTO 555
720            READ(22,130)(Y(J),J=1,DATANO)
C               WRITE(5,130)(Y(J),J=1,DATANO)
         GOTO 555
C
555      CONTINUE
         IF(ADDGRA.NE.'Y')GOTO 999
           IF(ICOUNT.GE.1)GOTO 444
              ICOUNT=ICOUNT+1
         DO 71 JC=1,DATANO
            YH(JC)=Y(JC)
71       CONTINUE
         CLOSE(UNIT=22)
C
         WRITE(5,365)
           READ(5,170)FILE1
             WRITE(5,82)
               READ(5,230)FORM
              WRITE(5,370)
             READ(5,170)FILE2
C
         OPEN(UNIT=22,STATUS='OLD',READONLY,FILE=FILE1,
         1      ACCESS='SEQUENTIAL')
         OPEN(UNIT=23,STATUS='NEW',FILE=FILE2,
         1      ACCESS='SEQUENTIAL')
           READ(22,180)DATANO
         GOTO 666
C
444      CONTINUE
         DO 73 JY=1,DATANO
             Y(JY)=YH(JY)+Y(JY)
73       CONTINUE
         WRITE(23,180)DATANO
              DO 74 JX=1,DATANO
                 WRITE(23,130)Y(JX)
74       CONTINUE
C
         CLOSE(UNIT=23)
C
999      CONTINUE
                 DO 2 J=1,DATANO
                  X(J)=INTER*REAL(J)
2                CONTINUE
C
                  CALL GRAPOL(X,Y,DATANO)
            CALL CHASIZ(1.5,2.5)
         CALL CHASTR(SITE)
C
              DO 3 K=1,DATANO-1
               ERF=.5*INTER*60.*(Y(K)+Y(K+1))
                SUM=SUM+ERF
3         CONTINUE
C
            CALL PENSEL(1,0.7,3)
         CALL CHASIZ(2.0,3.0)
             CALL GRAMOV(DURA*0.82,MAXF*(0.85-(REAL(I)-1.0)*0.05))
                 CALL CHAFIX(SUM,9,2)
             CALL CHAHOL(6H AT *.)
         CALL CHASTR(SITE)
C
         IF(SETT.NE.'Y')GOTO 23
         DO 19 ID=1,DATANO
           IF(Y(ID).LE.DWF)YD(ID)=Y(ID)
           IF(Y(ID).GT.DWF)YD(ID)=DWF
19       CONTINUE
C
         CALL GRAMOV(0.0,0.0)
            CALL GRAPOL(X,YD,DATANO)
            CALL CHASIZ(1.5,2.5)
         CALL CHASTR(XDWF)
```

```
C
          DO 21 K=1,DATANO-1
           ERF=.5*INTER*60.*(YD(K)+YD(K+1))
            TOT=TOT+ERF
21       CONTINUE
        DIFF=SUM-TOT
C
           CALL PENSEL(1,0.7,3)
        CALL CHASIZ(2.0,3.0)
            CALL GRAMOV(DURA*0.82,MAXF*(0.85-(REAL(I+1)-1.0)*0.05))
                CALL CHAFIX(TOT,9,2)
            CALL CHAHOL(7H FOR *.)
        CALL CHASTR(XDWF)
            CALL GRAMOV(DURA*0.60,MAXF*(0.85-(REAL(I+2)-1.0)*0.05))
                CALL CHAHOL(16HNETT RUNVOL = *.)
        CALL CHAFIX(DIFF,9,2)
C
23      CONTINUE
        CLOSE(UNIT=22)
1       CONTINUE
        CLOSE(UNIT=22)
C
12      FORMAT(/,1X,'DO YOU WANT TO PLOT SETTING (Y/N)    ',$)
16      FORMAT(/,1X,'INPUT VALUE OF THE SETTING (REAL NO) ',$)
17      FORMAT(/,1X,'INPUT LABEL FOR SETTING (MAX 12 CHAR) ',$)
82      FORMAT(/,1X,'FORMAT TYPE ',$)
83      FORMAT(/,1X,'INPUT FORMAT TYPE OF FILE (1,2,3)    ',$)
88      FORMAT(F7.2,8X,F14.6)
91      FORMAT(/,1X,'DATANO = ',I4)
100     FORMAT(//,1X,'INPUT NO. OF HYDROGRAPH (INTEGER)    ',$)
110     FORMAT(I2)
120     FORMAT(/,1X,'INPUT DURATION OF EVENT (REAL NO)   ',$)
130     FORMAT(F10.4)
135     FORMAT(15X,F14.5)
140     FORMAT(/,1X,'MAX FLOW IN CUMECS  (REAL NO)   ',$)
150     FORMAT(F10.4)
165     FORMAT(//,1X,'INPUT FILE NAME (MAX 12 CHAR)   ',$)
166     FORMAT(//,1X,'RAINFALL FILENAME (MAX 12 CHAR)  ',$)
160     FORMAT(/,1X,'FILENAME ',$)
170     FORMAT(A12)
180     FORMAT(I5)
195     FORMAT(/,1X,'TIME INTERVAL OF DATA (REAL NO)   ',$)
190     FORMAT(/,1X,'INTERVAL ',$)
200     FORMAT(//,1X,'WHAT IS THE EVENT DATE FOR REAL STORM, OR',$)
215     FORMAT(/,1X,'INPUT THE HYDROGRAPH TITLE (MAX 12 CHAR)   ',$)
210     FORMAT(/,1X,'TITLE ',$)
220     FORMAT(//,1X,'INPUT NO. OF HYETOGRAPH (1 OR 2)     ',$)
230     FORMAT(I1)
240     FORMAT(/,1X,'INPUT STORM DURATION IN MIN (REAL NO)   ',$)
250     FORMAT(/,1X,'MAX. INTENSITY IN MM/HR (REAL NO)   ',$)
265     FORMAT(/,1X,'INPUT RAINGAUGE SITE NAME (MAX 16 CHAR)   ',$)
260     FORMAT(/,1X,'R/G S/NAME ',$)
270     FORMAT(A16)
285     FORMAT(/,1X,'IS HYDROGRAPH OBSERVED, WASSP OR DUCTS  ?',$)
280     FORMAT(/,1X,'O,W OR D ',$)
290     FORMAT(5X,'CHOOSE & INPUT EITHER O,W,D RESPECTIVELY -- ',$)
24      FORMAT(/,1X,'IS HYETOGRAPH (A)1 OR (B)10 COLUMNS ?',$)
26      FORMAT(5X,'CHOOSE & INPUT EITHER A/B RESPECTIVELY -- ',$)
27      FORMAT(/,1X,'A OR B ',$)
300     FORMAT(A1)
310     FORMAT(8F10.4)
320     FORMAT(8F10.4)
330     FORMAT(10F10.2)
340     FORMAT(10F8.2)
350     FORMAT(5X,'RET PERIOD & DURATION FOR DESIGN (MAX 12 CHAR) -- ',$)
360     FORMAT(/,1X,'ADD 2 GRAPHS ? ',$)
365     FORMAT(/,1X,'ANOTHER FILENAME ',$)
370     FORMAT(/,1X,'OUTPUT FILENAME  ',$)
450     FORMAT(/,1X,'SCREEN PLOT OR PAPER PLOT, ENTER S OR P -- ',$)
500     FORMAT(/,1X,'ENTER THE LENGTH FOR X-AXIS (REAL NO) --- ',$)
501     FORMAT(/,1X,'ENTER THE LENGTH FOR Y-AXIS (REAL NO) --- ',$)
510     FORMAT(/,1X,'GRID LINE ALONG Y-AXIS ?   (YES OR NO) --- ',$)
520     FORMAT(F5.1)
10      FORMAT(////,15X,'*****************************************',$)
15      FORMAT(15X,'*                                       *',$)
20      FORMAT(15X,'* RAINFALL HYETOGRAPH & RUNOFF HYDROGRAPH *',$)
25      FORMAT(15X,'*                                       *',$)
30      FORMAT(15X,'*        GRAPHICAL PLOTTING PROGRAM     *',$)
35      FORMAT(15X,'*                                       *',$)
40      FORMAT(15X,'*      Dundee College of Technology     *',$)
45      FORMAT(15X,'*         Civil Engineering Dept.       *',$)
50      FORMAT(15X,'*     ( Urban Storm Drainage Group )    *',$)
55      FORMAT(15X,'*                                       *',$)
60      FORMAT(15X,'*****************************************',$)
65      FORMAT(/,1X,'   ',$)
11      FORMAT(//////,1X,'DO YOU WANT INFORMATION (Y/N)  ',$)
```

```
      DO 4 I=1,HYETO
        IF(SORP.EQ.'S')GOTO 85
       WRITE(5,166)
       READ(5,170)FILE1
        WRITE(5,24)
        WRITE(5,26)
        READ(5,300)CHOICE
          WRITE(5,195)
          READ(5,150)INTER
                WRITE(5,265)
                READ(5,270)RSITE
      GOTO 87
85     WRITE(5,160)
       READ(5,170)FILE1
        WRITE(5,27)
        READ(5,300)CHOICE
          WRITE(5,190)
          READ(5,150)INTER
                WRITE(5,260)
                READ(5,270)RSITE
87    CONTINUE
C
              CALL PENSEL(1,0.5,0)
C     X-AXIS POSITION, SCALE, ETC.
      IF(SORP.EQ.'P'.OR.SORP.EQ.'p')GOTO 622
C             CALL AXIPOS(1,20.0,YLEN+30.0+((REAL(I-1))*50.0),XLEN,1)
              CALL AXIPOS(1,20.0,YLEN+40.0+((REAL(I-1))*60.0),XLEN,1)
               GOTO 623
622           CALL AXIPOS(1,20.0,YLEN+((REAL(I))*60.0),XLEN,1)
623           CALL AXISCA(1,10,0.0,STDU,1)
              CALL AXIDRA(-1,1,1)
C     Y-AXIS POSITION, ETC.
      IF(SORP.EQ.'P'.OR.SORP.EQ.'p')GOTO 624
C             CALL AXIPOS(1,20.0,YLEN+30.0+((REAL(I-1))*50.0),40.0,2)
              CALL AXIPOS(1,20.0,YLEN+40.0+((REAL(I-1))*60.0),40.0,2)
                GOTO 625
624           CALL AXIPOS(1,20.0,YLEN+((REAL(I))*60.0),40.0,2)
625           CALL AXISCA(1,5,0.0,MAXIN,2)
              CALL AXIDRA(1,-1,2)
C     TITLE FOR Y-AXIS
      CALL HARCHA
         CALL CHAANG(90.0)
              CALL CHASIZ(2.0,3.0)
              CALL GRAMOV(DURA/-20.0,MAXIN*0.1)
         CALL CHAHOL(18HRAINFALL (MM/HR)*.)
      CALL CHAANG(0.0)
C
              OPEN(UNIT=22,STATUS='OLD',READONLY,FILE=FILE1,
     1             ACCESS='SEQUENTIAL')
               READ(22,180)DATANO
      IF(CHOICE.NE.'A')GOTO 28
               DO 5 J=1,DATANO
                READ(22,130)YV(J)
C               WRITE(5,130)YV(J)
5              CONTINUE
      GOTO 29
28    READ(22,340)(YV(J),J=1,DATANO)
29    CONTINUE
      INA=IFIX(INTER)
          IF(INA.EQ.1)GOTO 90
      GOTO 560
C     RE-ARRANGE RAINFALL DATA IF INTER IS NOT EQUAL TO 1
560   CONTINUE
      K=0
      DO 70 II=1,DATANO
      RDATA=YV(II)
          DO 75 JM=1,INA
              K=K+1
              YY(K)=RDATA
75    CONTINUE
70    CONTINUE
          DATPT=K
      GOTO 18
C
90    CONTINUE
              DO 95 IT=1,DATANO
                YY(IT)=YV(IT)
95            CONTINUE
      DATPT=DATANO
C
18    CONTINUE
C
C     open(unit=4,file='test',type='new')
C     DO 80 IX=1,DATPT
C         WRITE(4,*)YY(IX)
C80   CONTINUE
C
```

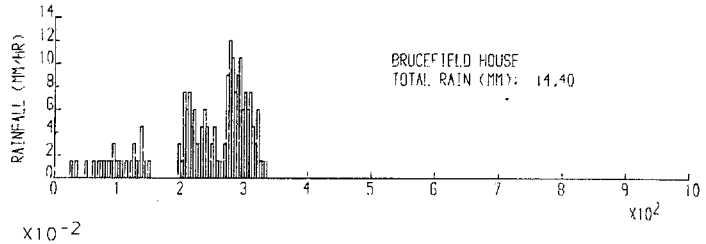
```
C   HISTOGRAM USING LINE DRAWING ROUTINES
        XHS=0.0
        DO 86 NH=1,DATANO
           CALL GRAMOV(XHS,0.0)
              CALL GRALIN(XHS,YV(NH))
          XHS=XHS+INTER
                CALL GRALIN(XHS,YV(NH))
           CALL GRALIN(XHS,0.0)
86      CONTINUE
C
        ERA=0.0
        SUN=0.0
        DO 7 L=1,DATANO
           ERA=YV(L)/60.*INTER
           SUN=SUN+ERA
7       CONTINUE
C    TITLE AND RAINFALL DEPTH FOR HYETOGRAPH
        CALL GRAMOV(STDU*.55,MAXIN*.7)
            CALL CHASTR(RSITE)
C
        CALL GRAMOV(STDU*.55,MAXIN*.58)
            CALL CHAHOL(19HTOTAL RAIN (MM): *.)
                CALL CHAFIX(SUN,6,2)
C
                CLOSE(UNIT=22)
4           CONTINUE
C
        CALL DEVEND
        CALL GINEND
        STOP
        END
C
C   ************************************************
        SUBROUTINE NEWS
C   ************************************************
        WRITE(5,21)
        WRITE(5,22)
        WRITE(5,23)
        WRITE(5,24)
        WRITE(5,26)
        WRITE(5,27)
        WRITE(5,28)
        WRITE(5,29)
        WRITE(5,31)
        WRITE(5,32)
        WRITE(5,33)
        WRITE(5,34)
        WRITE(5,36)
        WRITE(5,37)
C
21      FORMAT(//,1X,'This graphical plotting program utilises the GINO &
     1 GINOGRAF subroutines to plot')
22      FORMAT(1X,'rainfall hyetograph and runoff hydrograph.  User can p
     1lot as many hydrographs as')
23      FORMAT(1X,'he likes but a maximum of only two hyetographs can be
     1accommodated in the pre-')
24      FORMAT(1X,'defined plotting areas.  A series of questions and
     1options have to be answered')
26      FORMAT(1X,'first & the program will then read in the data from
     1the files which had already')
27      FORMAT(1X,'been prepared.  The first line in the plot files is
     1the total number of points')
28      FORMAT(1X,'going to be plotted. The program will then read data
     1which could be in different')
29      FORMAT(1X,'type of formats depending on the nature of the graphs
     1and they are summarised as')
31      FORMAT(1X,'the following :-')
32      FORMAT(8X,'(a) One Single Column Datafile -- for hyetograph & old
     1 type of hydrograph')
33      FORMAT(8X,'(b) Three Columns Datafile -- for files processed by H
     1YDROMASTER but')
34      FORMAT(24X,'still required minor editing on flowrate')
36      FORMAT(8X,'(c) Three Columns Datafile -- for files processed by H
     1YDROMASTER and no')
37      FORMAT(24X,'more editing required')
        RETURN
        END
```

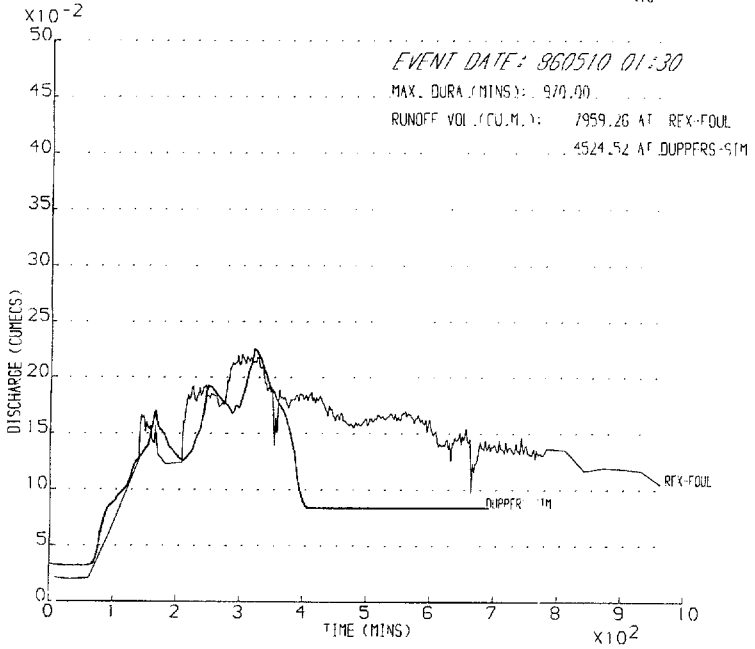
APPENDIX C

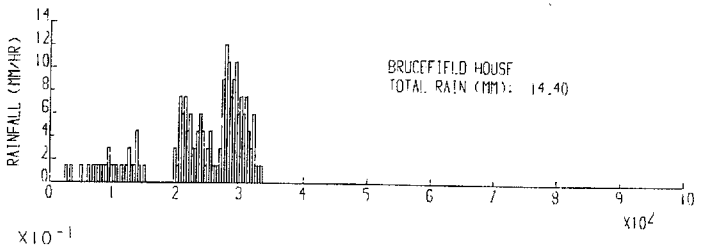
COMPARISON OF OBSERVED AND PREDICTED FLOWS (BY DUPPERS)

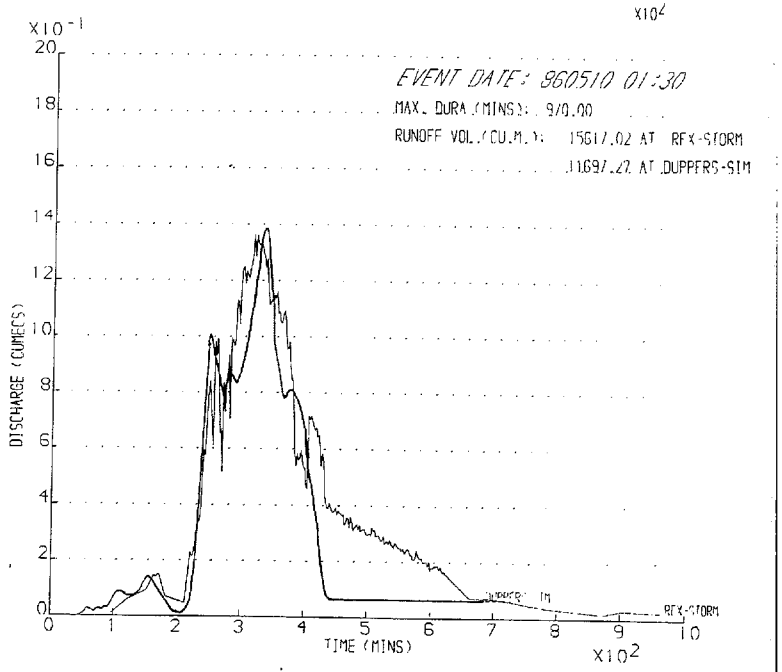
FOR PARALLEL PIPES AT CATCHMENT OUTFALLS

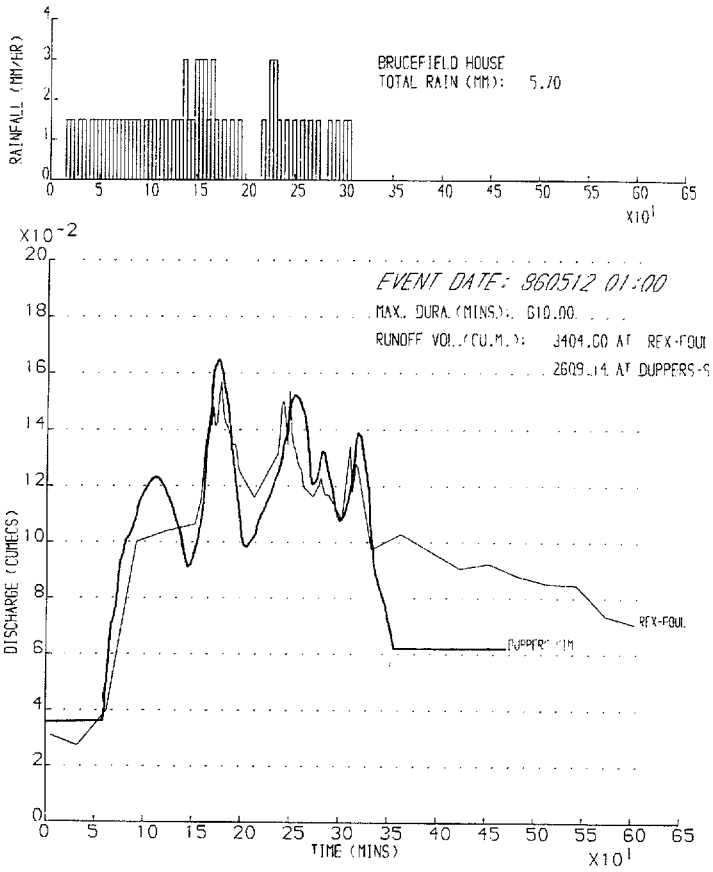
Note : Two pages of hydrographs are plotted for the parallel pipes separately, i.e. one for the foul and one for the strom relief, for each event.

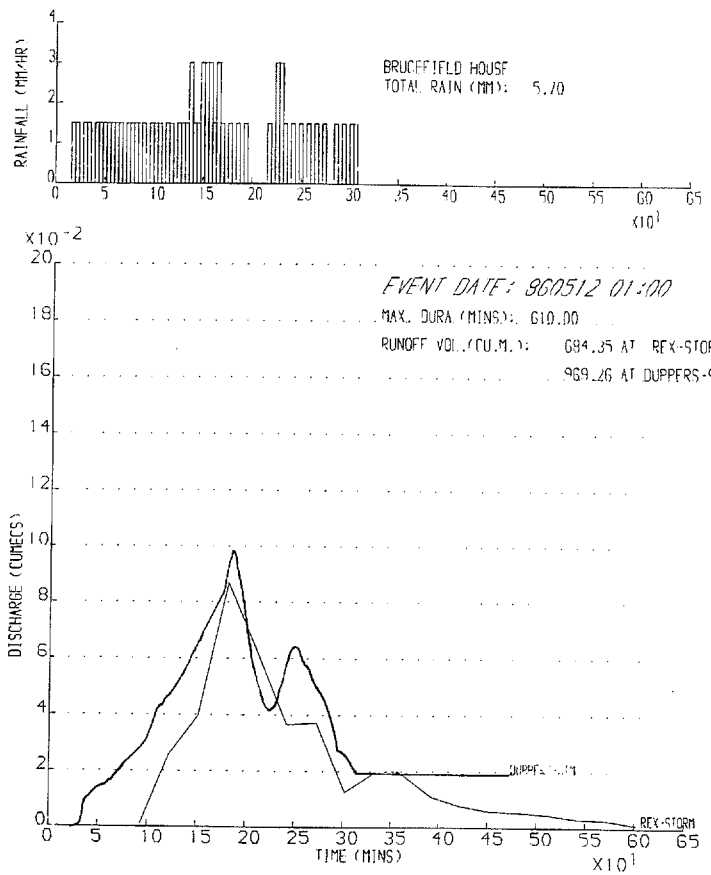

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 14.40
X10²

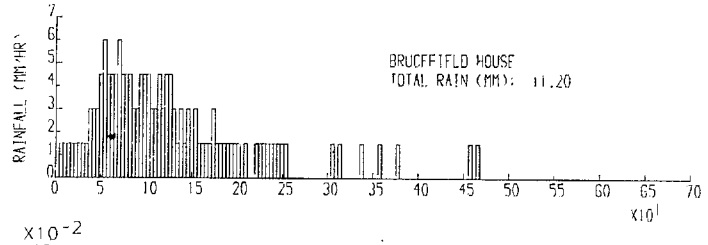

X10⁻²
EVENT DATE: 960510 01:30
MAX. DURA.(MINS): 970.00
RUNOFF VOL.(CU.M.): 7959.26 AT RFX-FOUL
4524.52 AT DUPPERS-SIM
DISCHARGE (CUMECS)
RFX-FOUL
DUPPER 1M
TIME (MINS)
X10²

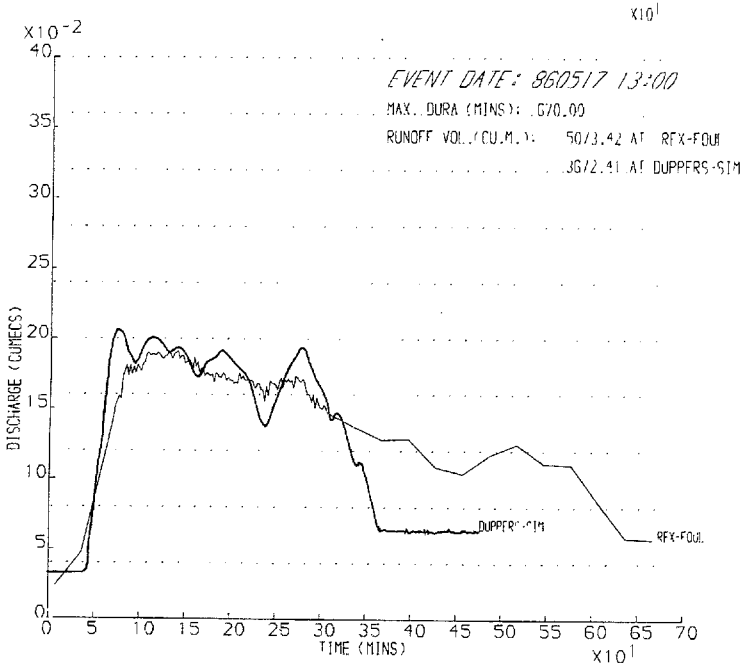

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 14.40
X10²

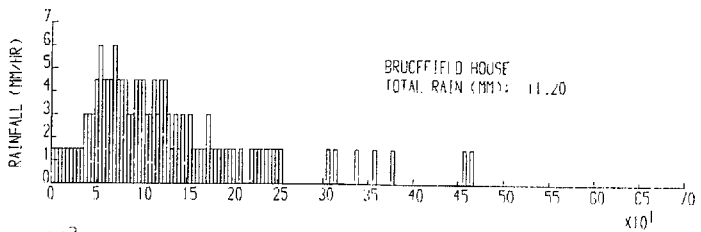

X10⁻¹
EVENT DATE: 960510 01:30
MAX. DURA.(MINS): 970.00
RUNOFF VOL.(CU.M.): 15617.02 AT RFX-STORM
11697.22 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPER 1M
RFX-STORM
TIME (MINS)
X10²

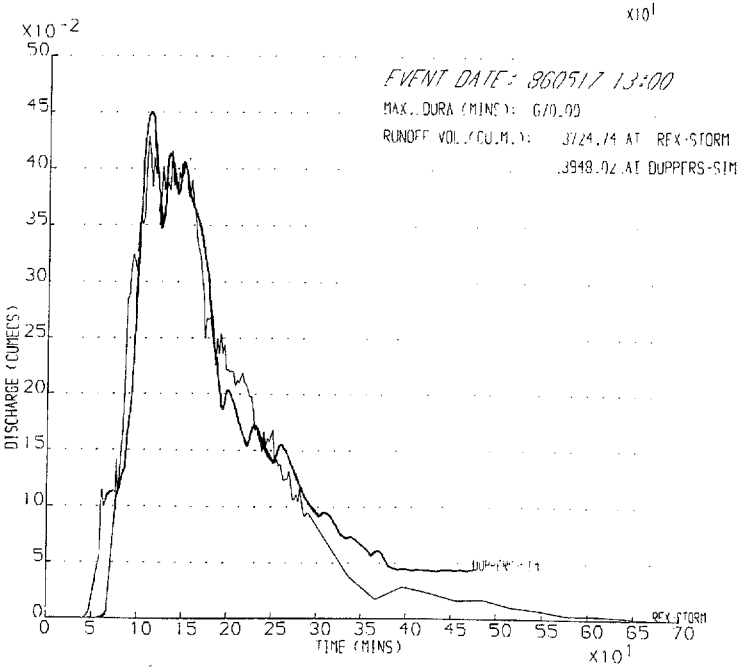

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 5.70
X10
X10 -2
EVENT DATE: 860512 01:00
MAX. DURA (MINS): 610.00
RUNOFF VOL. (CU.M.): 3404.60 AT REX-FOUL
2609.14 AT DUPPERS-SIM
DISCHARGE (CUMECS)
REX-FOUL
DUPPERS-SIM
TIME (MINS)
X10

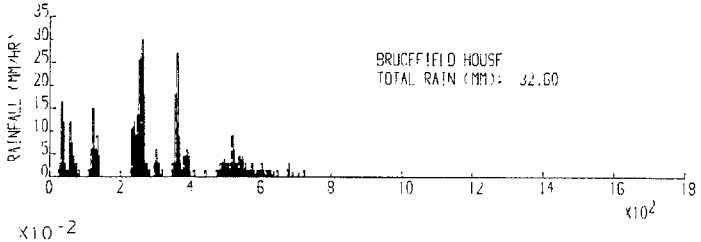

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 5.70
X10
X10 -2
EVENT DATE: 860512 01:00
MAX. DURA (MINS): 610.00
RUNOFF VOL. (CU.M.): 694.35 AT REX-STORM
969.26 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
REX-STORM
TIME (MINS)
X10

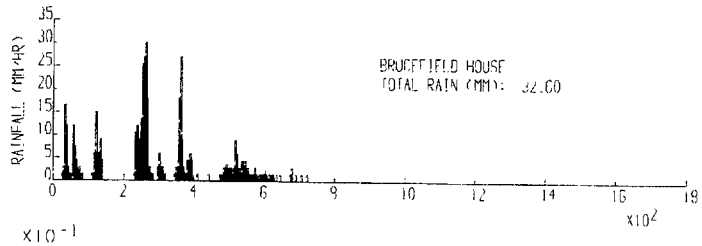

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 11.20
X10^1

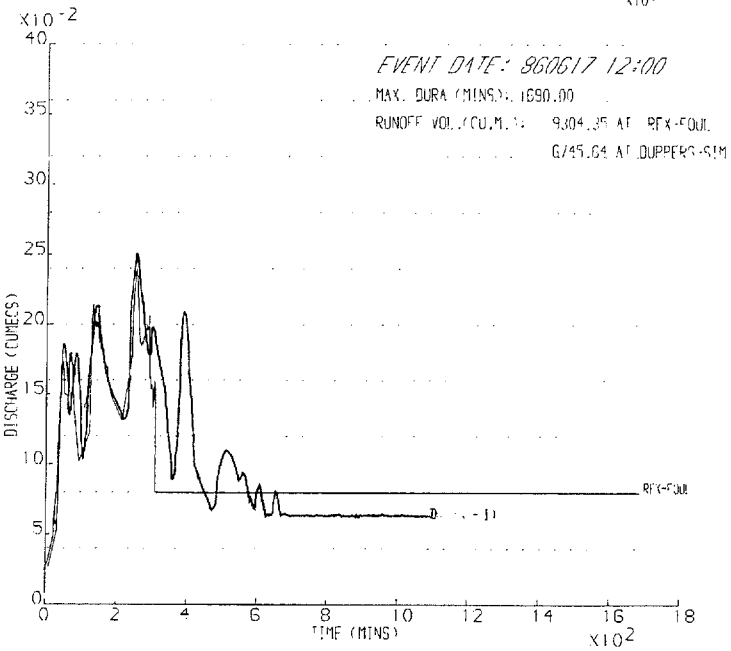

X10^-2
EVENT DATE: 860517 13:00
MAX. DURA (MINS): 670.00
RUNOFF VOL.(CU.M.): 5073.42 AT RFX-FOUR
3672.41 AT DUPPERS-SIM
DISCHARGE (CUMECS)
TIME (MINS)
X10^1
DUPPERS-SIM
RFX-FOUR

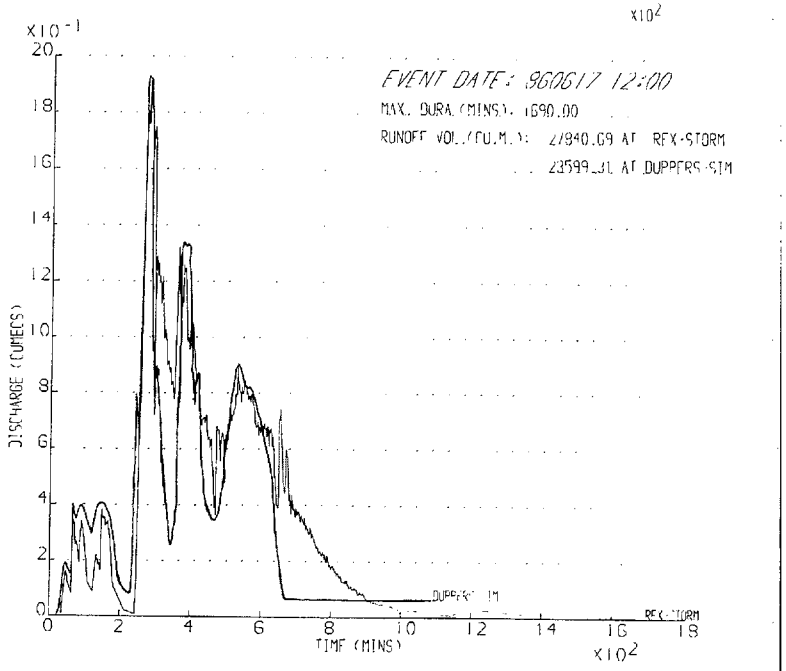

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 11.20
X10^1

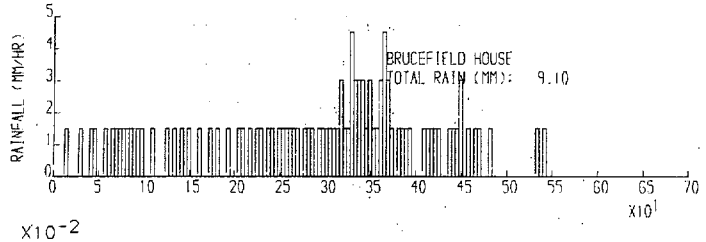

X10^-2
EVENT DATE: 860517 13:00
MAX. DURA (MINS): 670.00
RUNOFF VOL.(CU.M.): 3724.74 AT RFX-STORM
3948.02 AT DUPPERS-SIM
DISCHARGE (CUMECS)
TIME (MINS)
X10^1
RFX-STORM

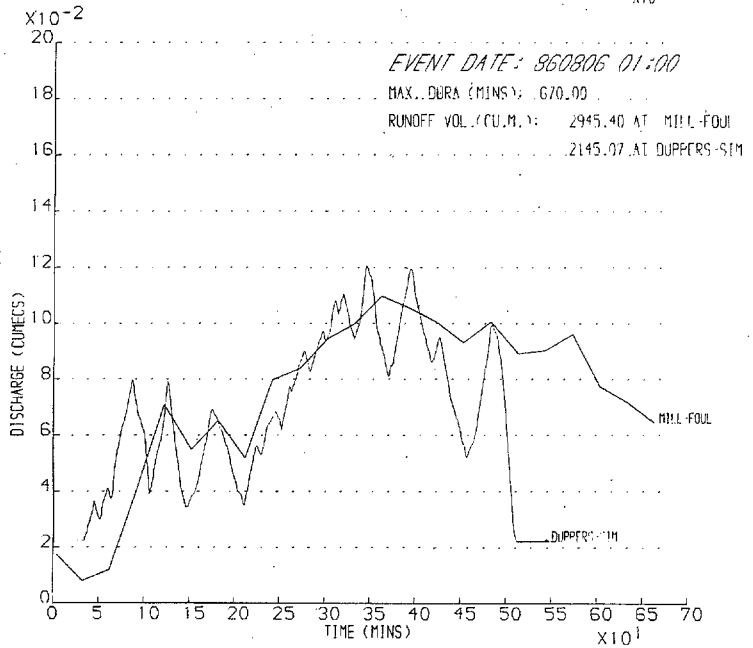

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 32.60
X10²

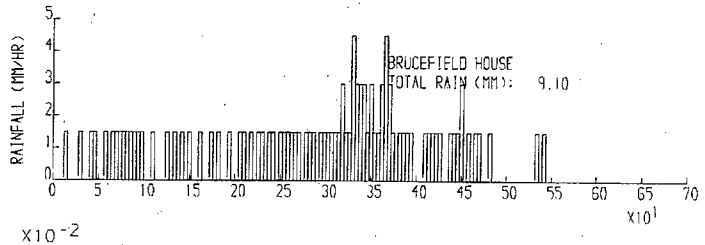

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 32.60
X10²

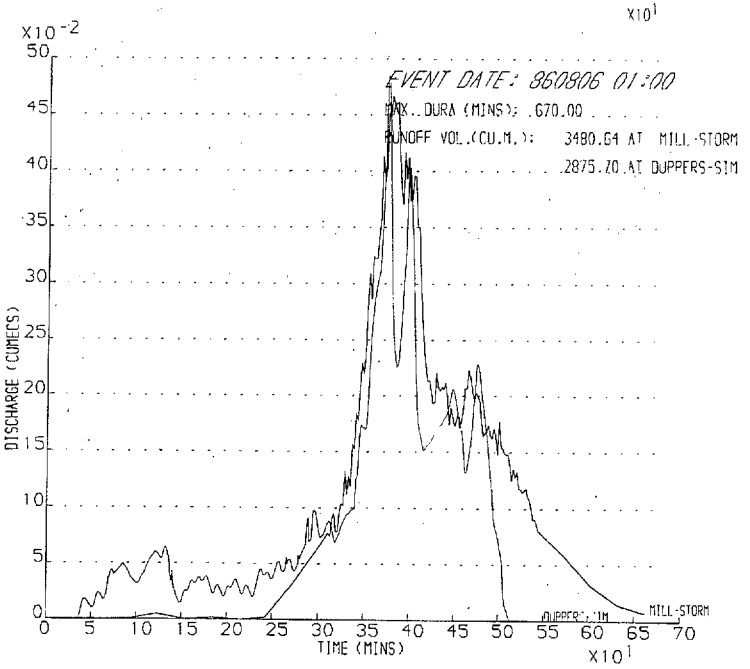

X10⁻²
EVENT DATE: 960617 12:00
MAX. DURA (MINS): 1690.00
RUNOFF VOL./CU.M.): 9304.35 AT REX-FOUL
6745.64 AT DUPPERS-SIM
DISCHARGE (CUMECS)
TIME (MINS)
X10²

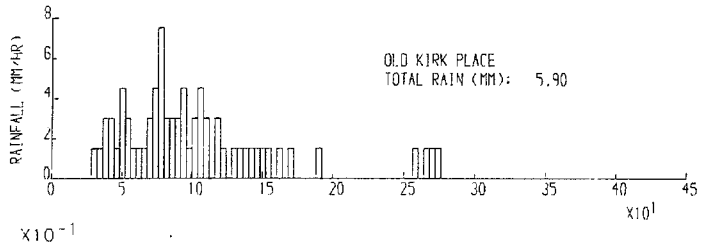

X10⁻¹
EVENT DATE: 960617 12:00
MAX. DURA (MINS): 1690.00
RUNOFF VOL./CU.M.): 27940.09 AT REX-STORM
23599.31 AT DUPPERS-SIM
DISCHARGE (CUMECS)
TIME (MINS)
X10²

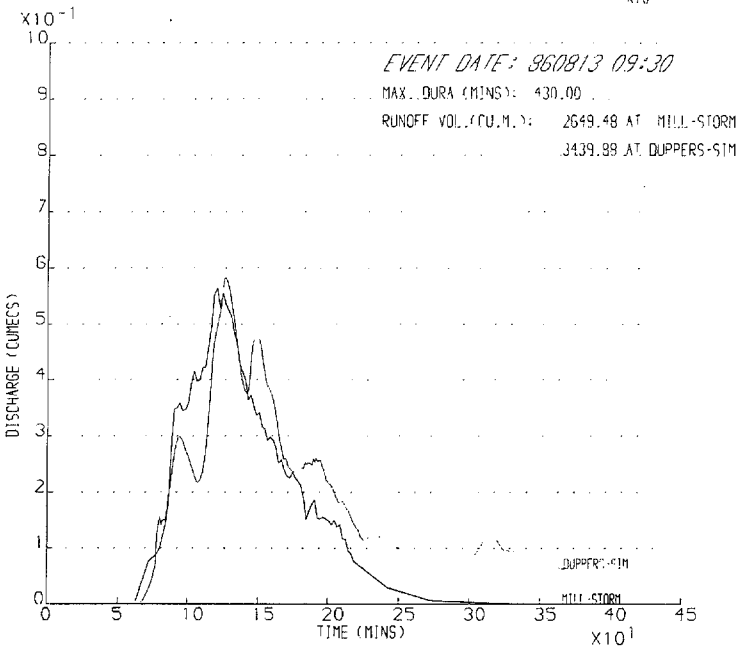

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 9.10
X10¹

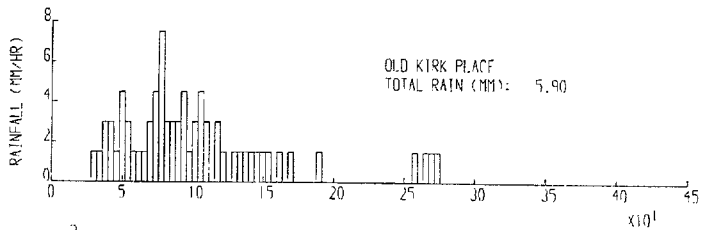

X10⁻²
EVENT DATE: 860806 01:00
MAX..DURA (MINS): 670.00
RUNOFF VOL.(CU.M.): 2945.40 AT MILL-FOUL
2145.07 AT DUPPERS-SIM
DISCHARGE (CUMECS)
MILL-FOUL
DUPPERS-SIM
TIME (MINS)
X10¹

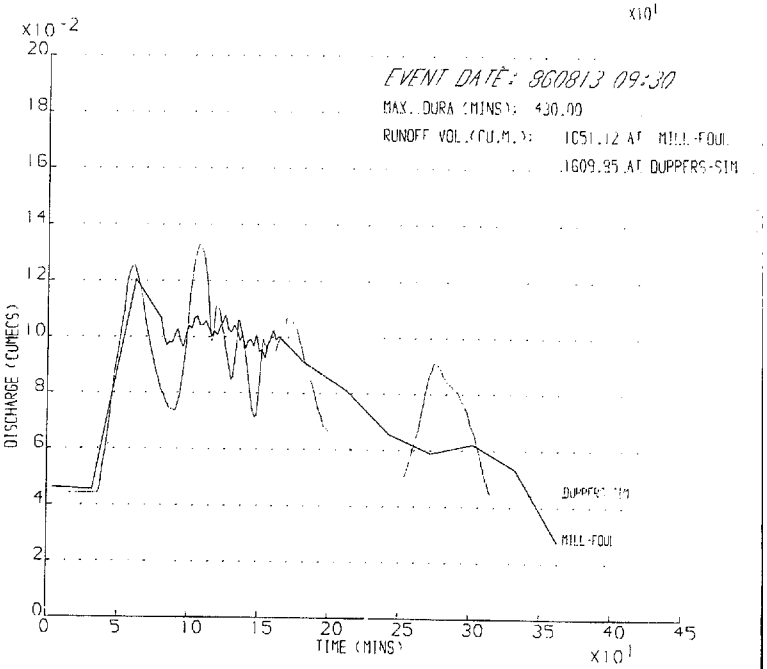

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 9.10
X10¹

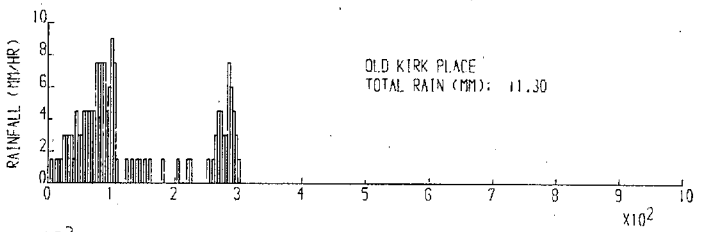

X10⁻²
EVENT DATE: 860806 01:00
MAX..DURA (MINS): 670.00
RUNOFF VOL.(CU.M.): 3480.64 AT MILL-STORM
2875.20 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
MILL-STORM
TIME (MINS)
X10¹

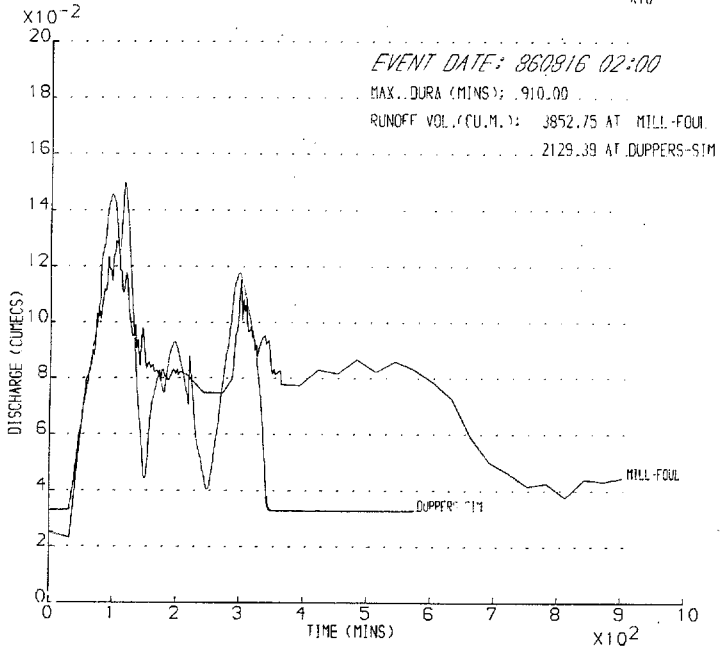

RAINFALL (MM/HR)
OLD KIRK PLACE
TOTAL RAIN (MM): 5.90
X10¹

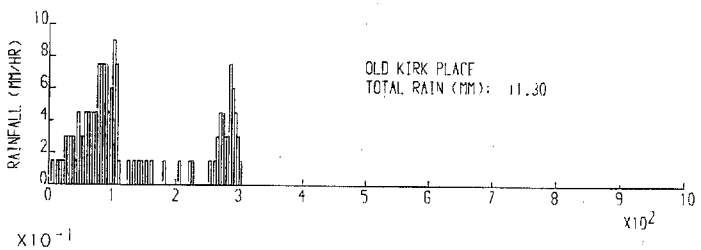

X10⁻¹
EVENT DATE: 960813 09:30
MAX. DURA (MINS): 430.00
RUNOFF VOL.(CU.M.): 2649.48 AT MILL-STORM
3439.89 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
MILL-STORM
TIME (MINS)
X10¹

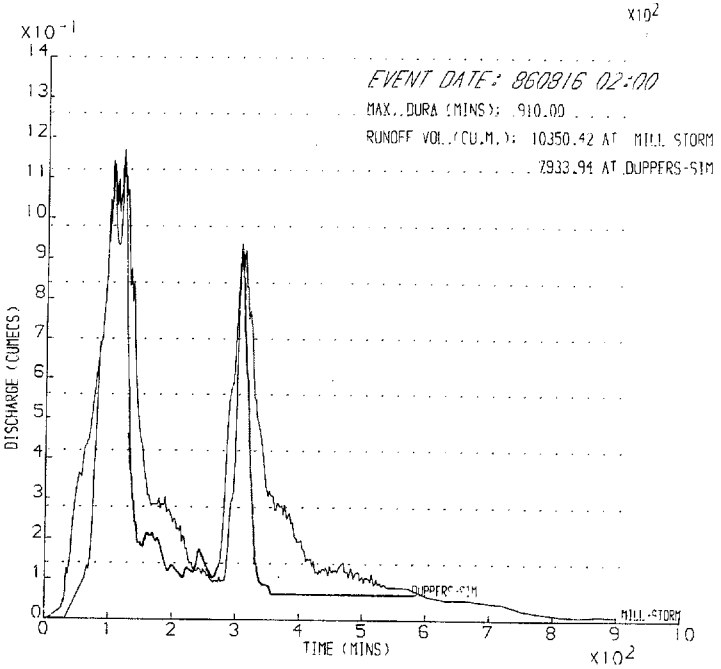

RAINFALL (MM/HR)
OLD KIRK PLACE
TOTAL RAIN (MM): 5.90
X10¹

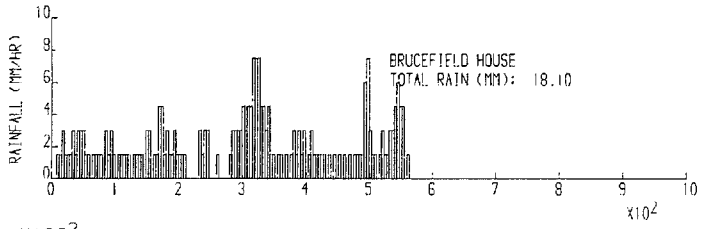

X10⁻²
EVENT DATE: 960813 09:30
MAX. DURA (MINS): 430.00
RUNOFF VOL.(CU.M.): 1651.12 AT MILL-FOUL
1609.35 AT DUPPERS-SIM
DISCHARGE (CUMECS)
MILL-FOUL
TIME (MINS)
X10¹

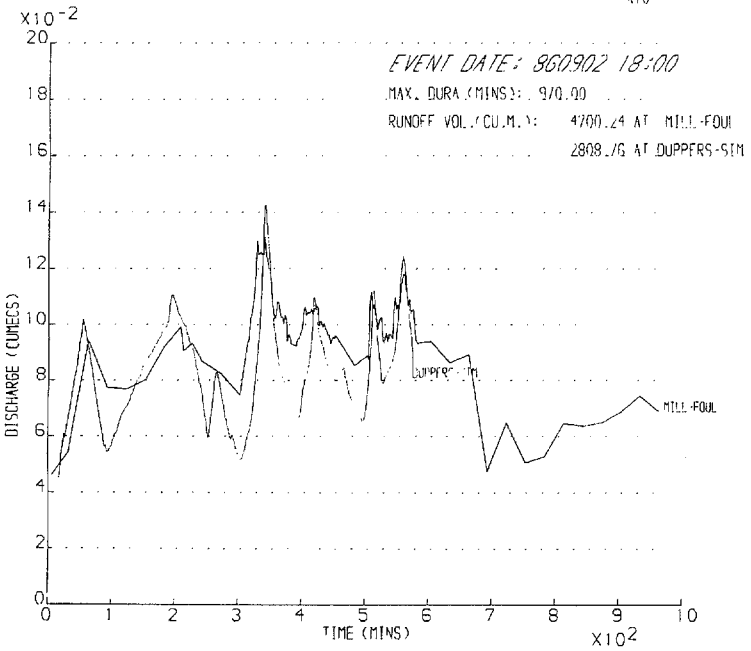

RAINFALL (MM/HR)
OLD KIRK PLACE
TOTAL RAIN (MM): 11.30
X10²

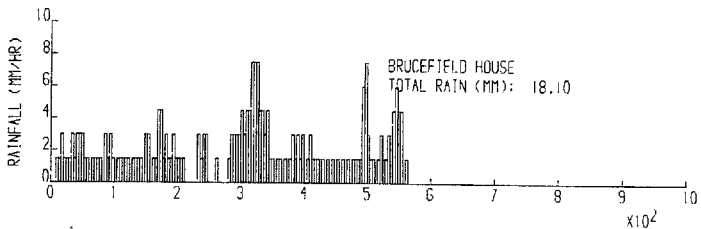

X10⁻²
EVENT DATE: 860816 02:00
MAX. DURA (MINS): 910.00
RUNOFF VOL. (CU.M.): 3852.75 AT MILL-FOUL
2129.39 AT DUPPERS-SIM
DISCHARGE (CUMECS)
MILL-FOUL
DUPPERS-SIM
TIME (MINS)
X10²

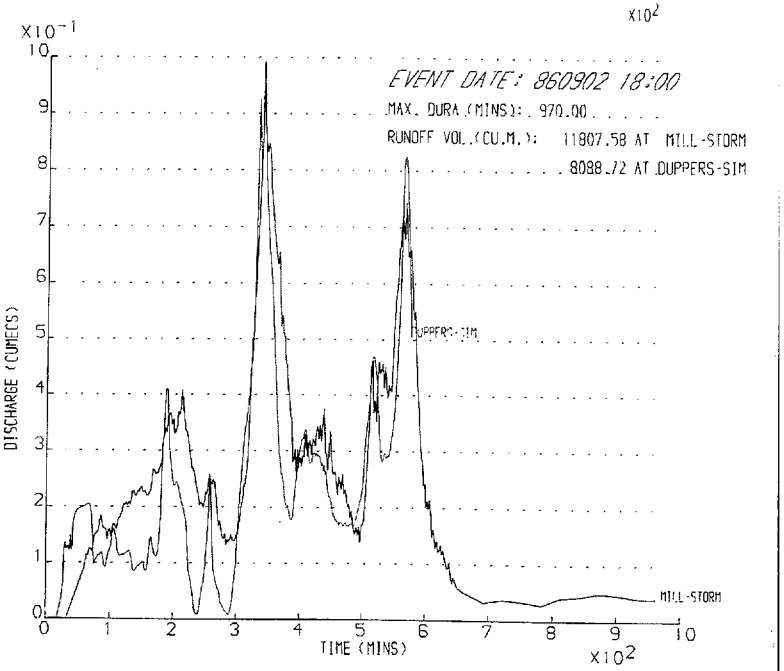

RAINFALL (MM/HR)
OLD KIRK PLACE
TOTAL RAIN (MM): 11.30
X10²

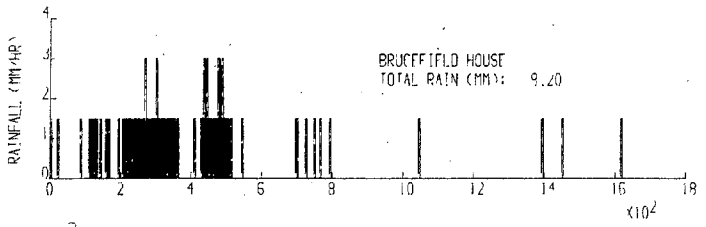

X10⁻¹
EVENT DATE: 860816 02:00
MAX. DURA (MINS): 910.00
RUNOFF VOL. (CU.M.): 10350.42 AT MILL STORM
7933.94 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
MILL-STORM
TIME (MINS)
X10²

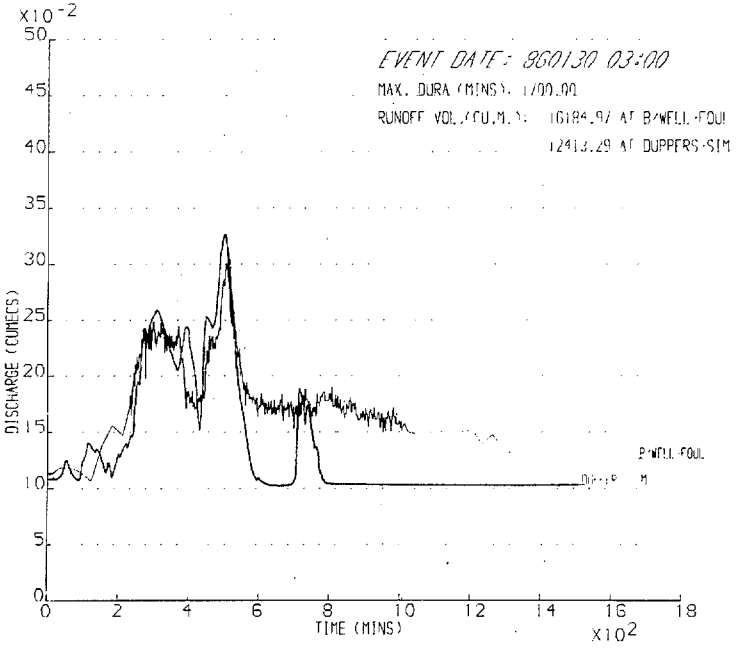

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 18.10
X10²

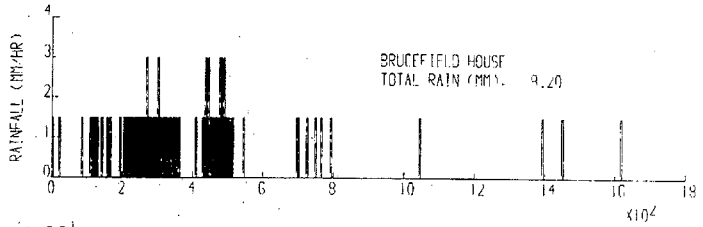

X10⁻²
EVENT DATE: 860902 18:00
MAX. DURA.(MINS): 970.00
RUNOFF VOL.(CU.M.): 4700.24 AT MILL-FOUL
2808.76 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
MILL-FOUL
TIME (MINS)
X10²

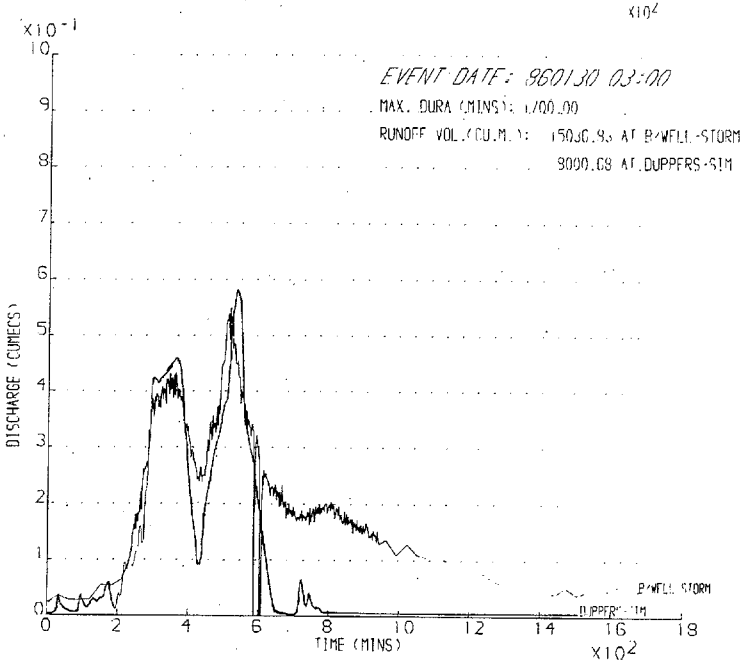

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 18.10
X10²

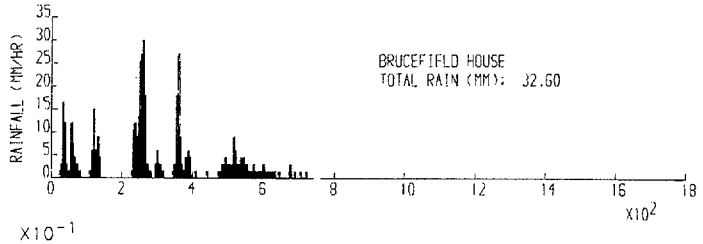

X10⁻¹
EVENT DATE: 860902 18:00
MAX. DURA.(MINS): 970.00
RUNOFF VOL.(CU.M.): 11807.58 AT MILL-STORM
8088.72 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
MILL-STORM
TIME (MINS)
X10²

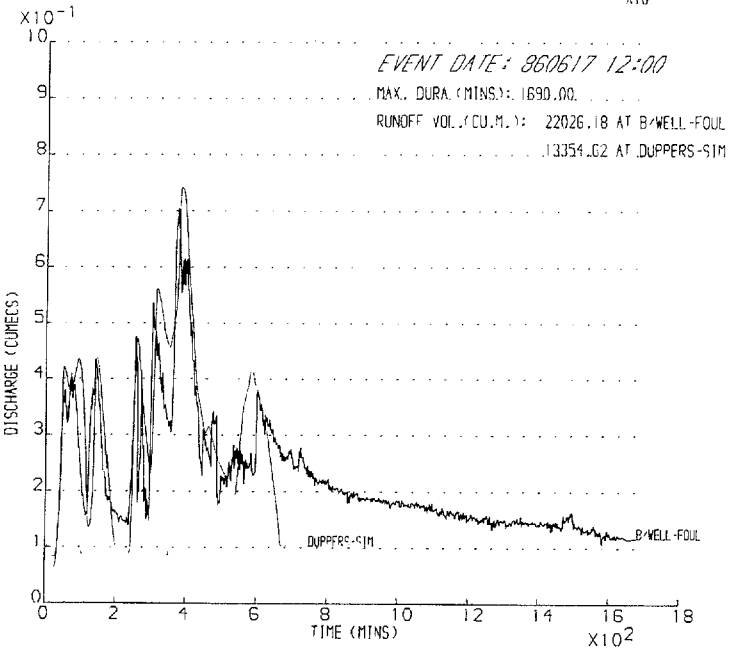

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 9.20
X10²

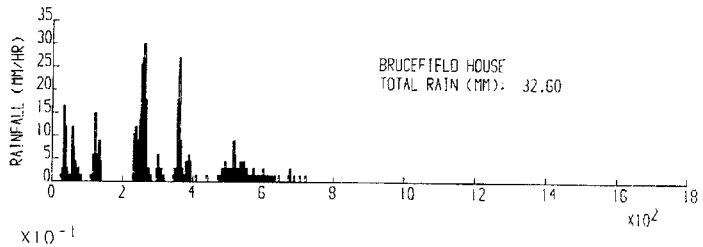

X10⁻²
EVENT DATE: 860130 03:00
MAX. DURA (MINS): 1700.00
RUNOFF VOL.(CU.M.): 16184.97 AT B'WELL-FOUL
12413.29 AT DUPPERS-SIM
DISCHARGE (CUMECS)
B'WELL FOUL
TIME (MINS)
X10²

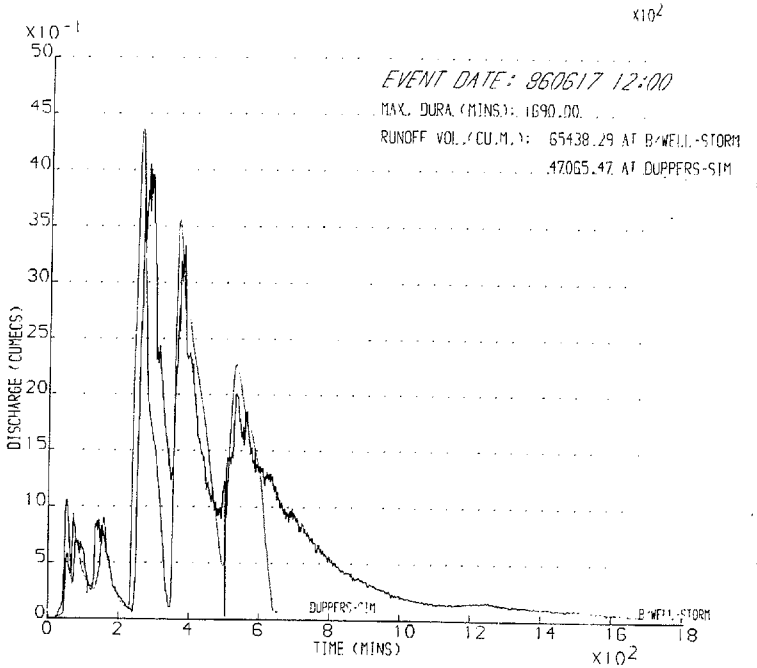

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 9.20
X10²

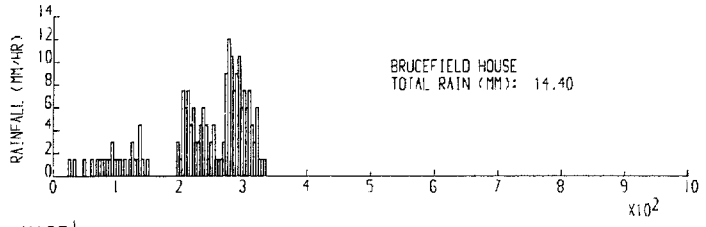

X10⁻¹
EVENT DATE: 860130 03:00
MAX. DURA (MINS): 1700.00
RUNOFF VOL.(CU.M.): 15036.95 AT B'WELL-STORM
9000.68 AT DUPPERS-SIM
DISCHARGE (CUMECS)
B'WELL STORM
TIME (MINS)
X10²

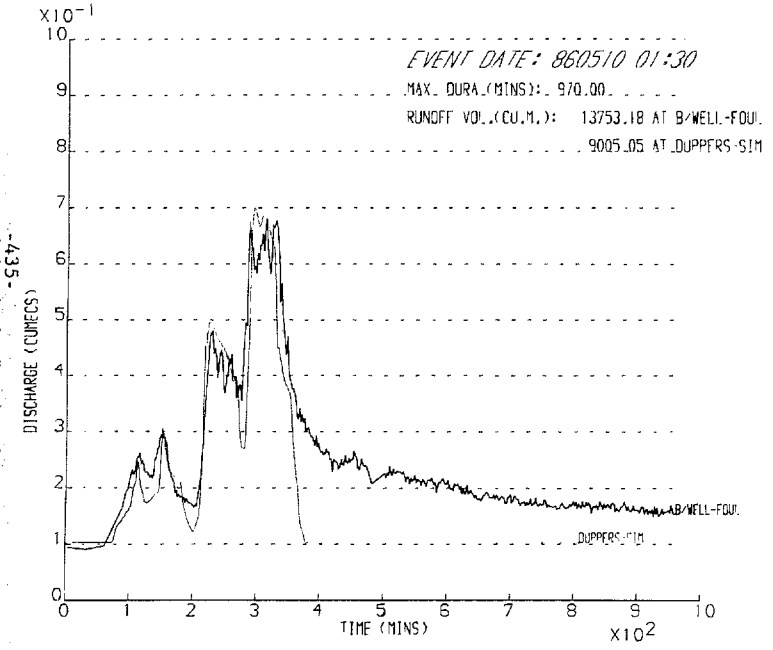

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 32.60
X10²

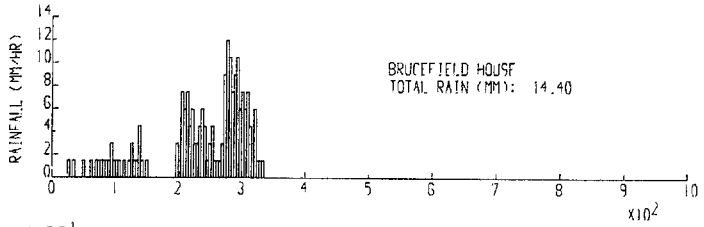

X10⁻¹
EVENT DATE: 860617 12:00
MAX. DURA.(MINS.): 1690.00
RUNOFF VOL.(CU.M.): 22026.18 AT B'WELL-FOUL
13354.02 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
B'WELL-FOUL
TIME (MINS)
X10²

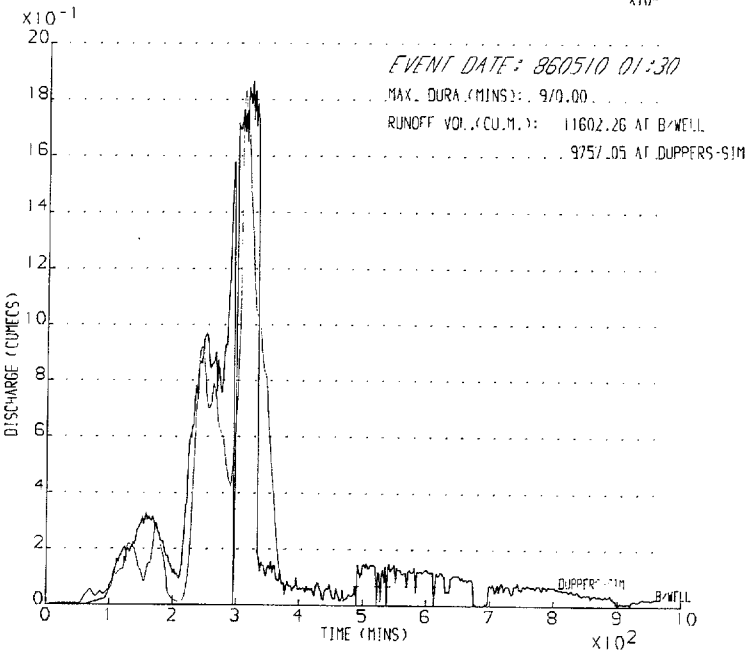

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 32.60
X10²

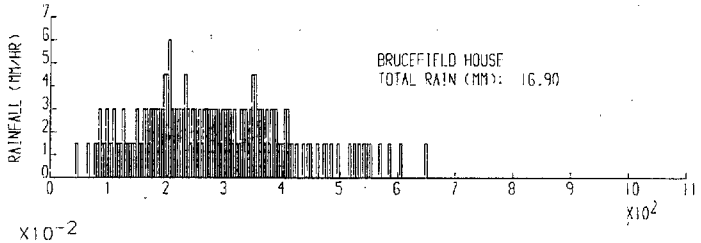

X10⁻¹
EVENT DATE: 860617 12:00
MAX. DURA.(MINS.): 1690.00
RUNOFF VOL.(CU.M.): 65438.29 AT B'WELL-STORM
47065.47 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
B'WELL-STORM
TIME (MINS)
X10²

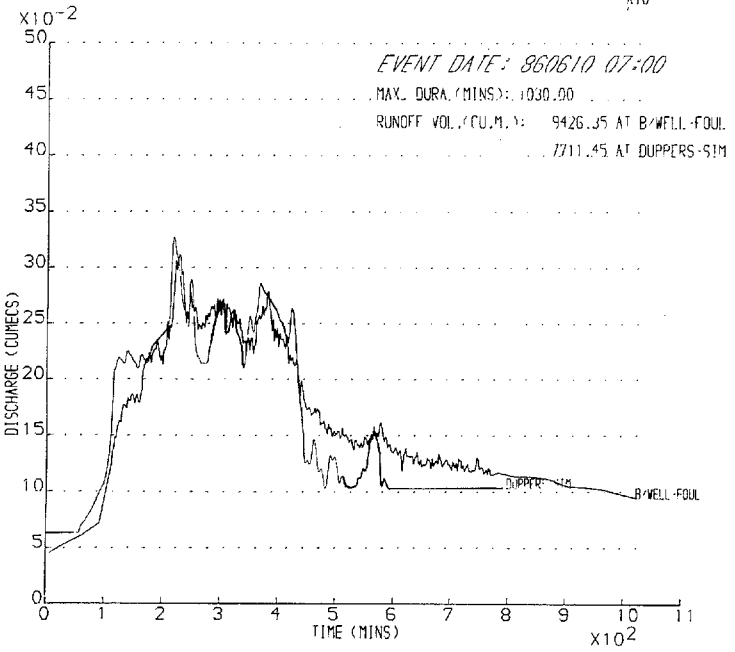

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 14.40
X10²

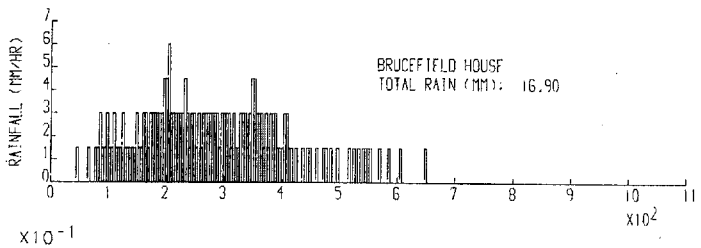

X10⁻¹
EVENT DATE: 860510 01:30
MAX. DURA.(MINS): 970.00
RUNOFF VOL.(CU.M.): 13753.18 AT B/WELL-FOUL
9005.05 AT DUPPERS-SIM
DISCHARGE (CUMECS)
AB/WELL-FOUL
DUPPERS-SIM
TIME (MINS)
X10²

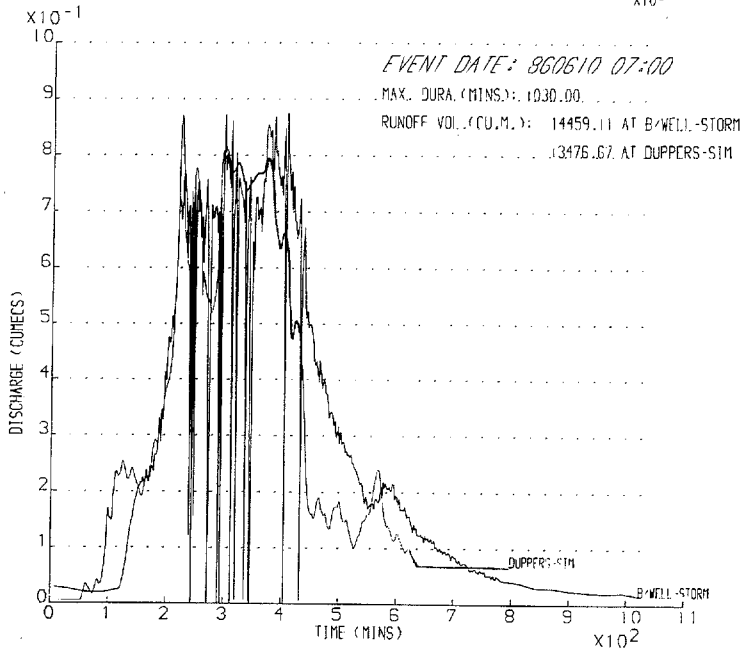

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 14.40
X10²

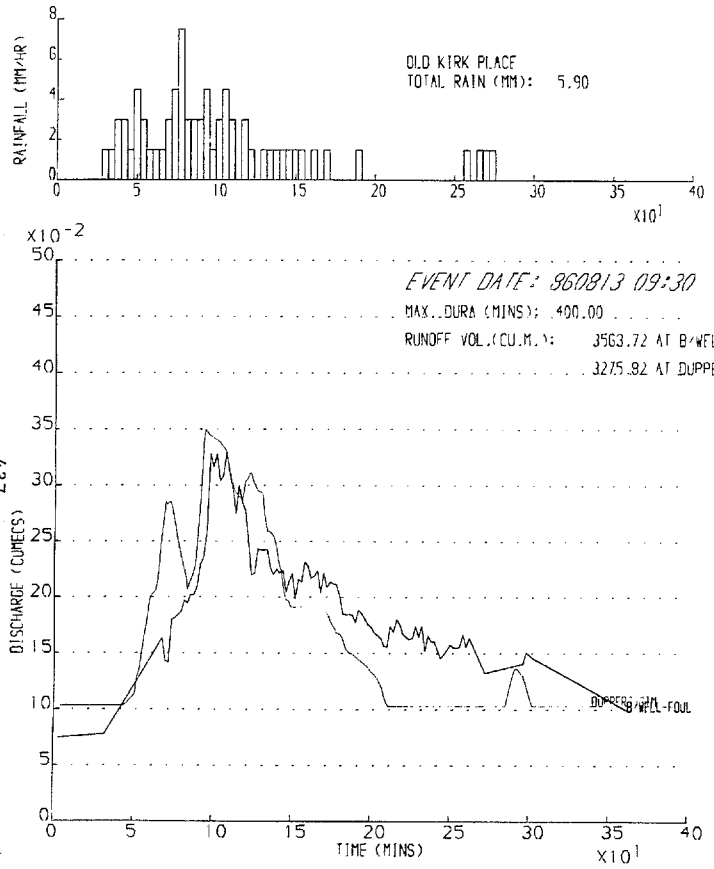

X10⁻¹
EVENT DATE: 860510 01:30
MAX. DURA.(MINS): 970.00
RUNOFF VOL.(CU.M.): 11602.26 AT B/WELL
9757.05 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
B/WELL
TIME (MINS)
X10²

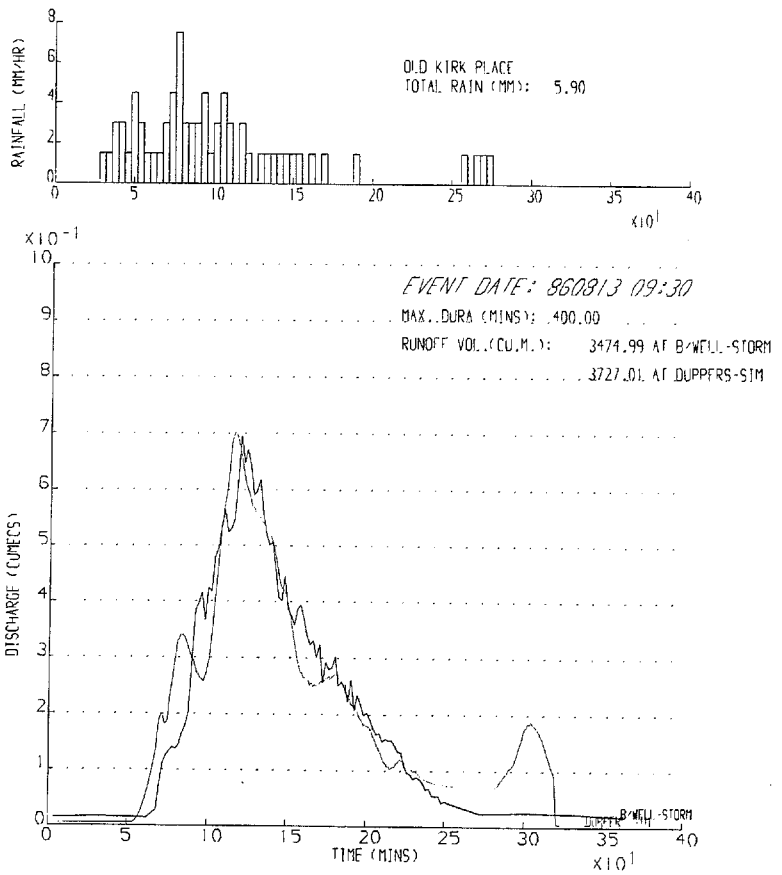

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 16.90
X10²

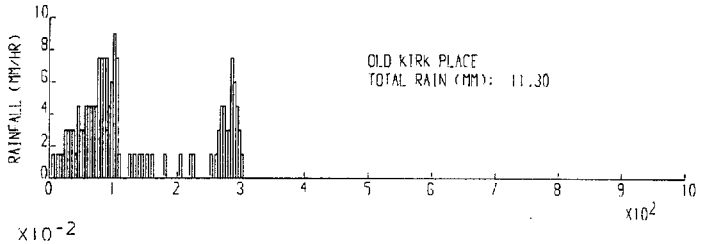

X10⁻²
EVENT DATE: 860610 07:00
MAX. DURA.(MINS): 1030.00
RUNOFF VOL.(CU.M.): 9426.35 AT B'WELL-FOUL
7711.45 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
B'WELL-FOUL
TIME (MINS)
X10²

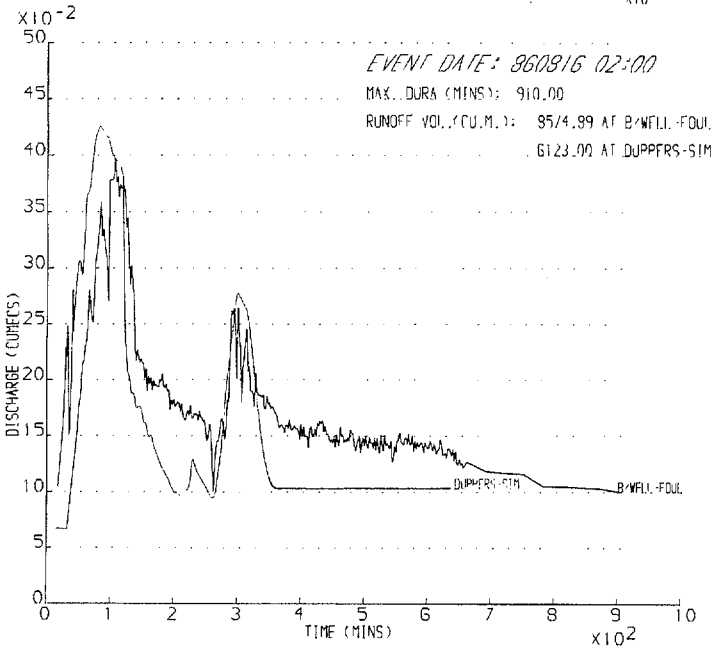

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 16.90
X10²

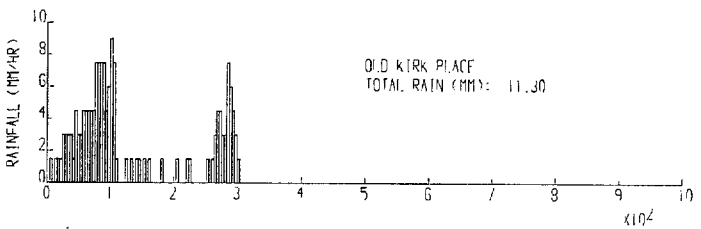

X10⁻¹
EVENT DATE: 860610 07:00
MAX. DURA.(MINS): 1030.00
RUNOFF VOL.(CU.M.): 14459.11 AT B'WELL-STORM
13476.67 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
B'WELL-STORM
TIME (MINS)
X10²

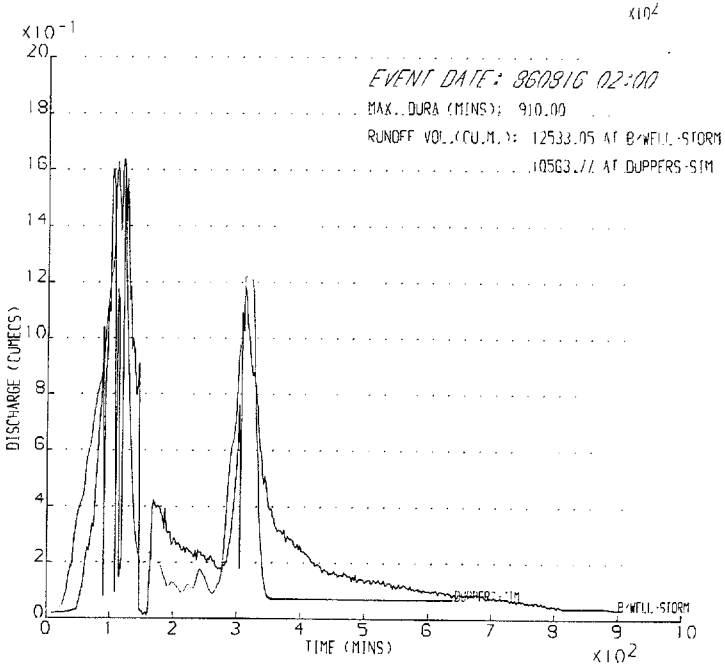

RAINFALL (MM/HR)
OLD KIRK PLACE
TOTAL RAIN (MM): 5.90
X10¹
X10⁻²
EVENT DATE: 860813 09:30
MAX..DURA (MINS): 400.00
RUNOFF VOL.(CU.M.): 3563.72 AT B/WFLL-FOUL
3275.82 AT DUPPERS-SIM
DISCHARGE (CUMECS)
TIME (MINS)
DUPPERS-SIM
B/WFLL-FOUL

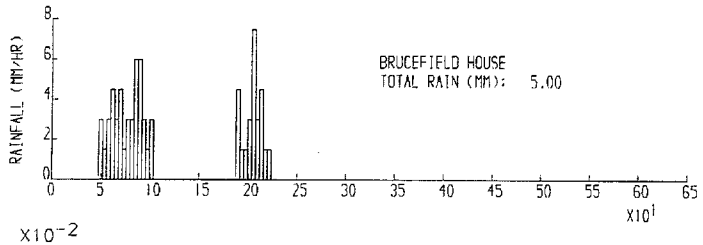

RAINFALL (MM/HR)
OLD KIRK PLACE
TOTAL RAIN (MM): 5.90
X10¹
X10⁻¹
EVENT DATE: 860813 09:30
MAX..DURA (MINS): 400.00
RUNOFF VOL.(CU.M.): 3474.99 AT B/WFLL-STORM
3727.01 AT DUPPERS-SIM
DISCHARGE (CUMECS)
TIME (MINS)
B/WFLL-STORM
DUPPERS-SIM

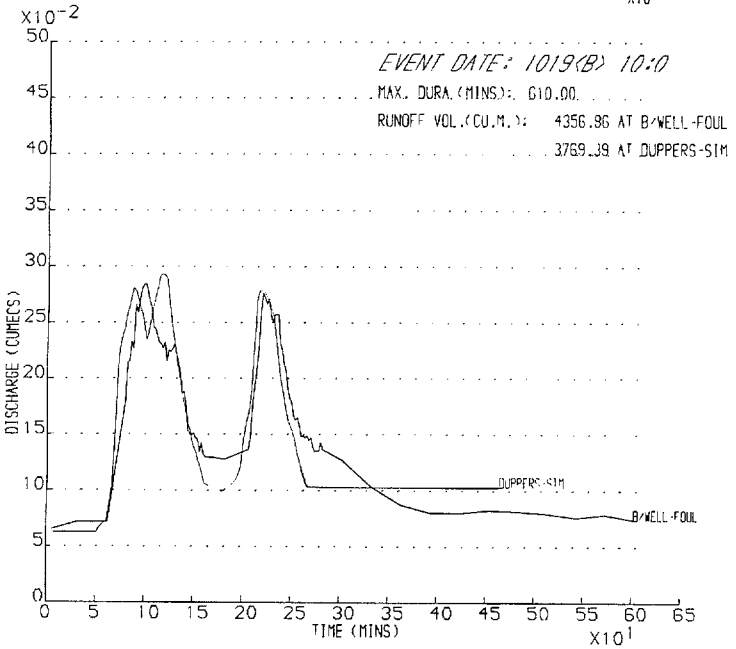

RAINFALL (MM/HR)
OLD KIRK PLACE
TOTAL RAIN (MM): 11.30
X10²

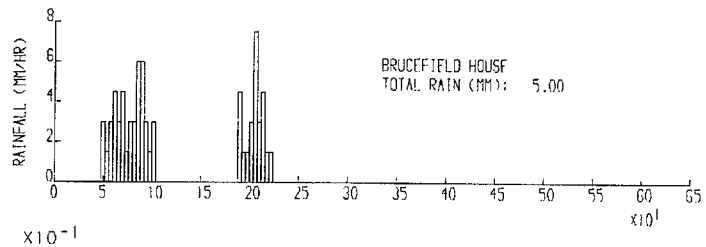

X10⁻²
EVENT DATE: 860816 02:00
MAX. DURA (MINS): 910.00
RUNOFF VOL.(CU.M.): 8574.99 AT B'WELL-FOUL
6123.00 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
B'WELL-FOUL
TIME (MINS)
X10²

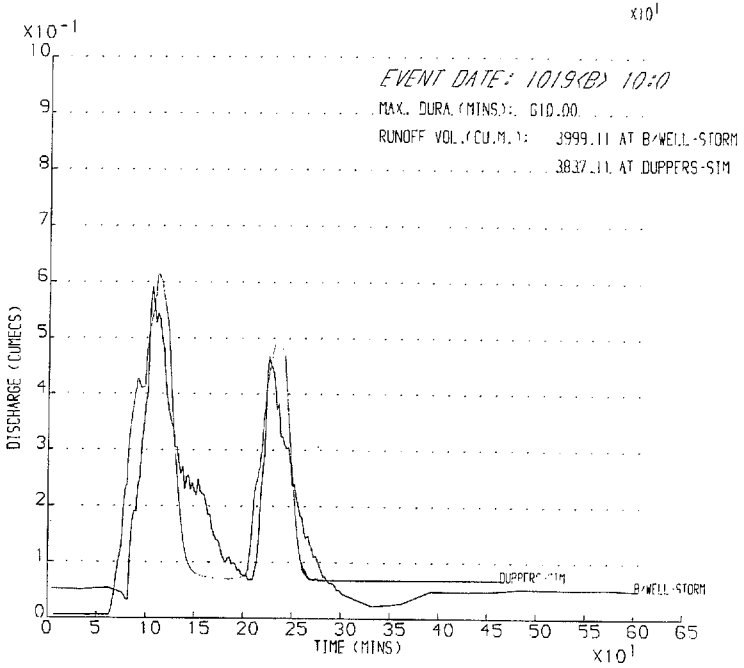

RAINFALL (MM/HR)
OLD KIRK PLACE
TOTAL RAIN (MM): 11.30
X10²

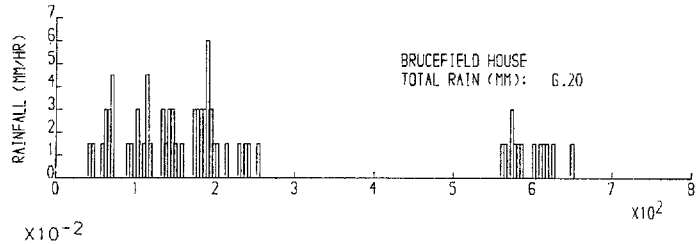

X10⁻¹
EVENT DATE: 860816 02:00
MAX. DURA (MINS): 910.00
RUNOFF VOL.(CU.M.): 12533.05 AT B'WELL-STORM
10563.77 AT DUPPERS-SIM
DISCHARGE (CUMECS)
B'WELL-STORM
TIME (MINS)
X10²

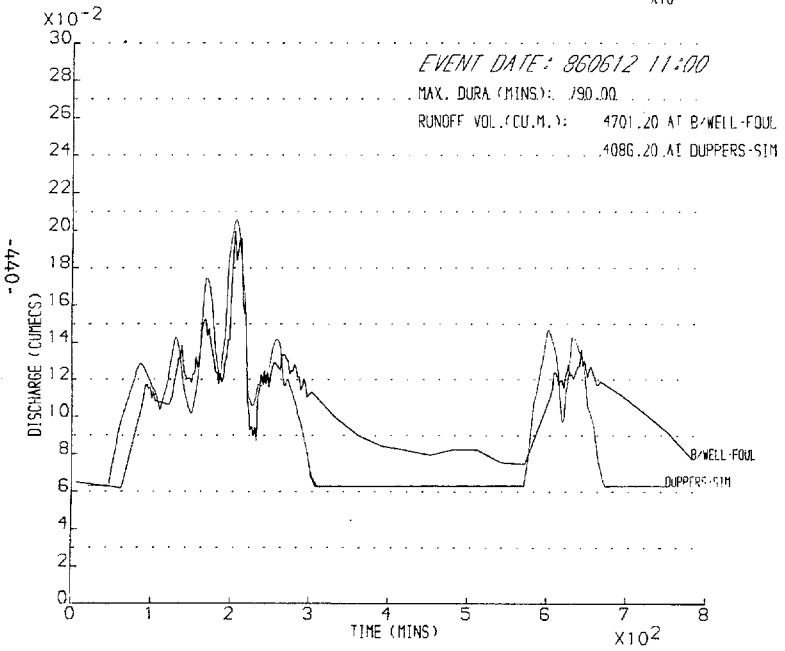

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 5.00
X10¹

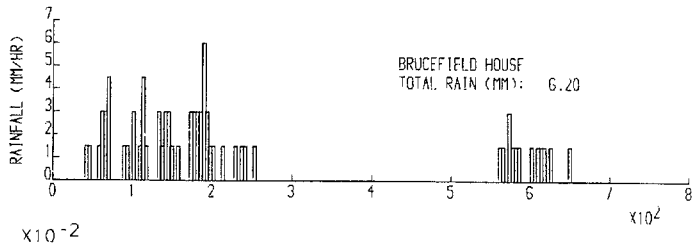

X10⁻²
EVENT DATE: 1019(B) 10:0
MAX. DURA.(MINS.): 610.00
RUNOFF VOL.(CU.M.): 4356.86 AT B'WELL-FOUL
3769.39 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
B'WELL-FOUL
TIME (MINS)
X10¹

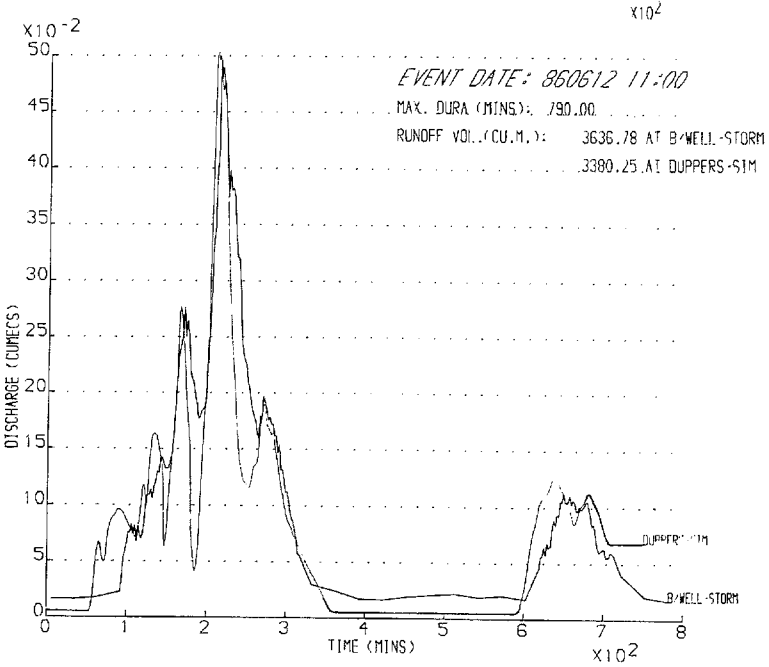

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 5.00
X10¹

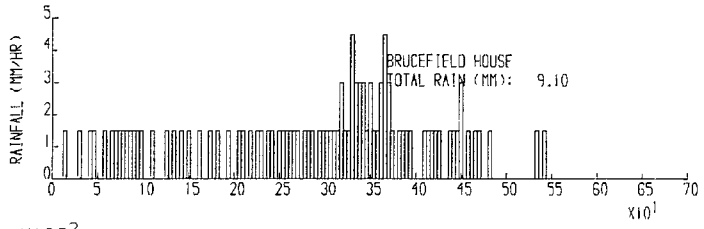

X10⁻¹
EVENT DATE: 1019(B) 10:0
MAX. DURA.(MINS.): 610.00
RUNOFF VOL.(CU.M.): 3999.11 AT B'WELL-STORM
3837.11 AT DUPPERS-SIM
DISCHARGE (CUMECS)
B'WELL-STORM
TIME (MINS)
X10¹


RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 6.20
X10²

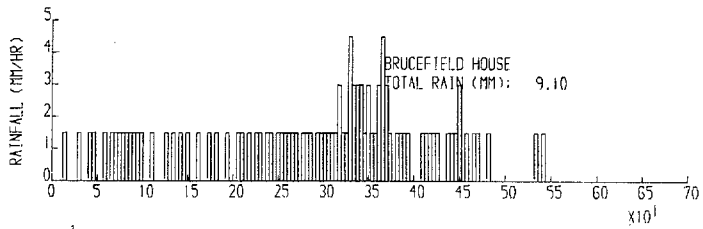

X10⁻²
EVENT DATE: 860612 11:00
MAX. DURA (MINS): 790.00
RUNOFF VOL.(CU.M.): 4701.20 AT B/WELL-FOUL
4086.20 AT DUPPERS-SIM
DISCHARGE (CUMECS)
B/WELL-FOUL
DUPPERS-SIM
TIME (MINS)
X10²

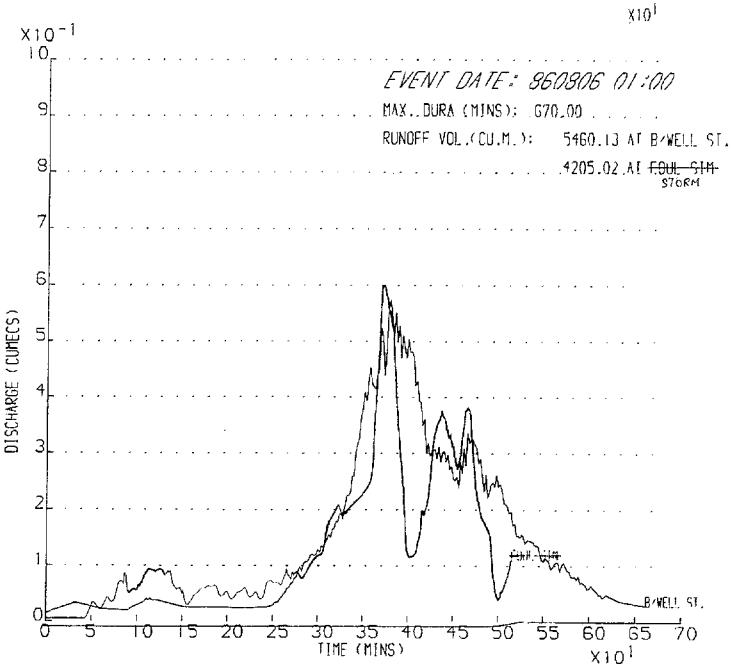

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 6.20
X10²

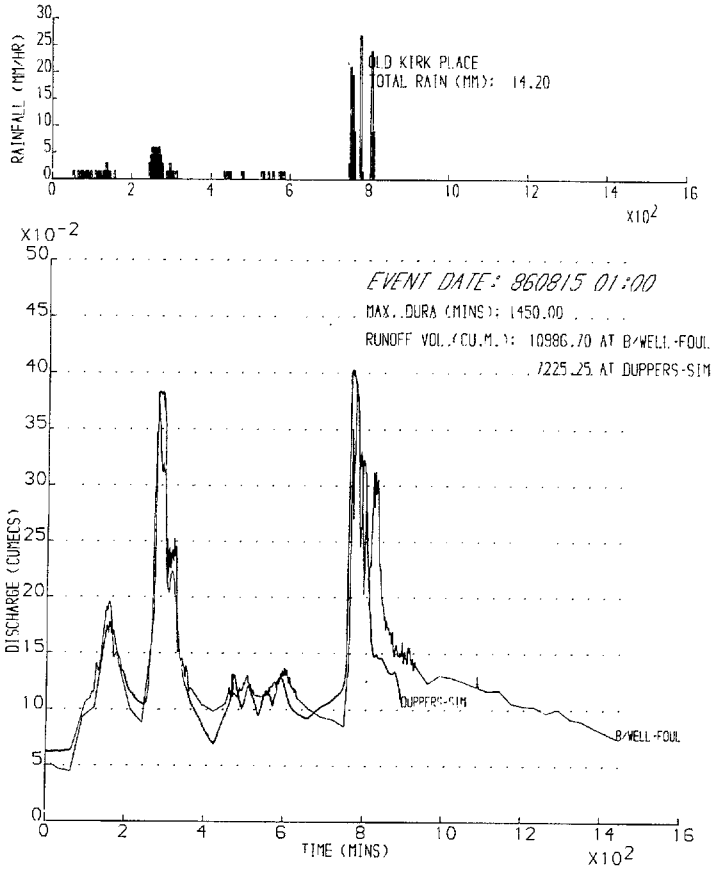

X10⁻²
EVENT DATE: 860612 11:00
MAX. DURA (MINS): 790.00
RUNOFF VOL.(CU.M.): 3636.78 AT B/WELL-STORM
3380.25 AT DUPPERS-SIM
DISCHARGE (CUMECS)
B/WELL-STORM
TIME (MINS)
X10²

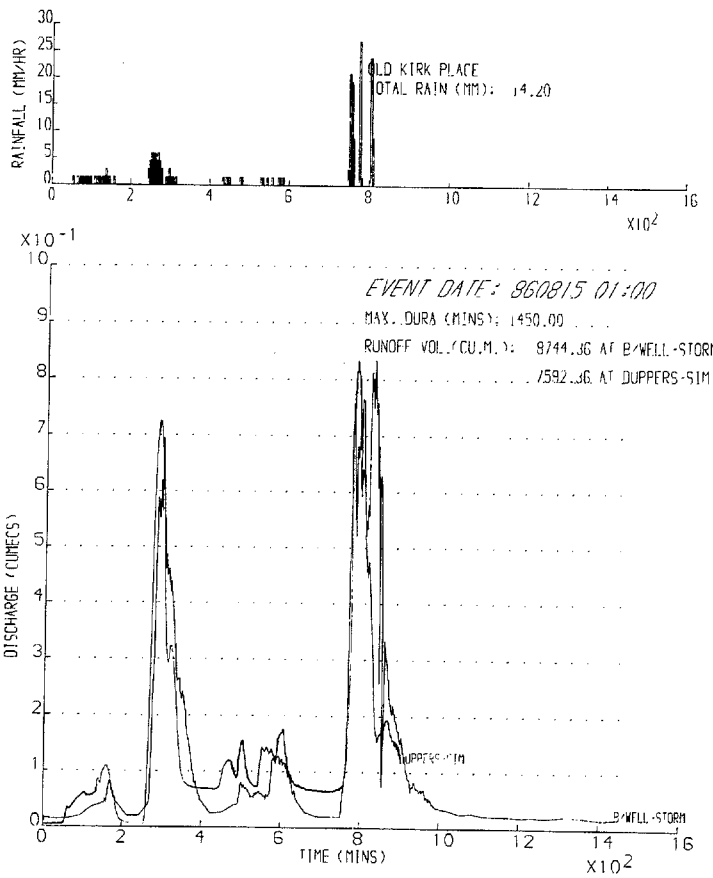

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 9.10
X10¹

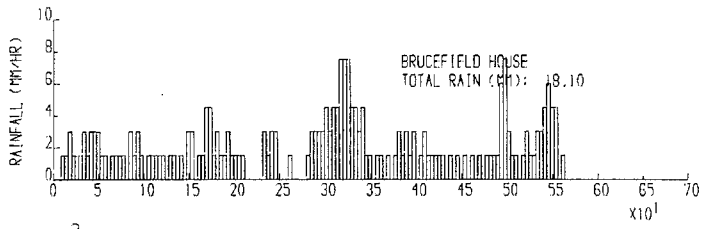

X10⁻²
EVENT DATE: 860806 01:00
MAX..DURA (MINS): 670.00
RUNOFF VOL.(CU.M.): 6054.94 AT B/WELL(FOUL)
4257.99 AT DUPPERS-SIM
DISCHARGE (CUMECS)
B/WELL(FOUL)
TIME (MINS)
X10¹

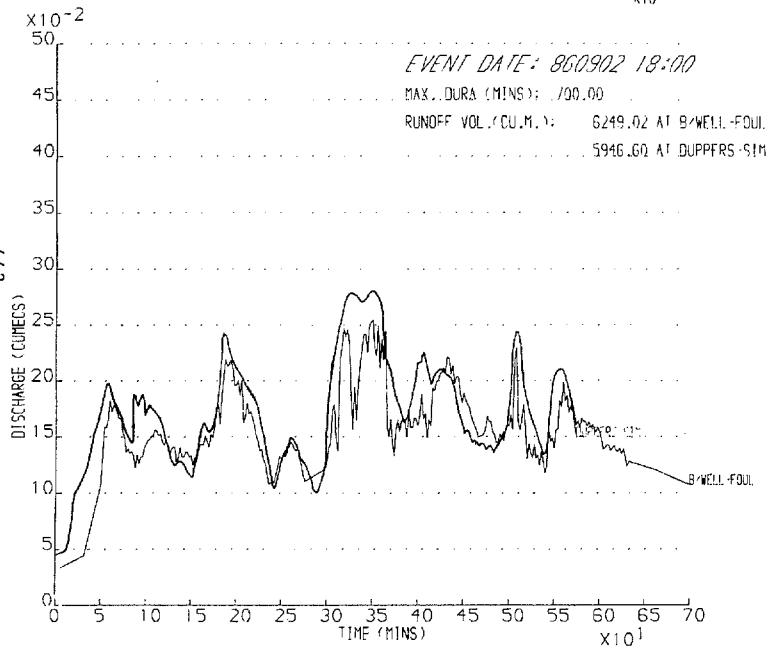

RAINFALL (MM/HR)
BRUCEFIELD HOUSE
TOTAL RAIN (MM): 9.10
X10¹

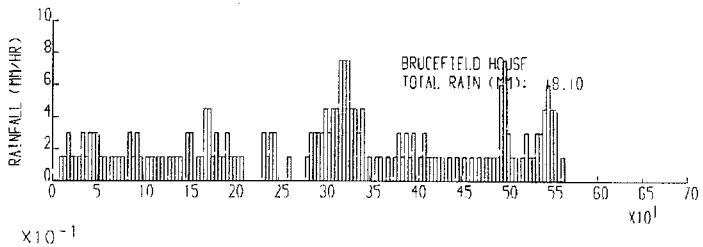

X10⁻¹
EVENT DATE: 860806 01:00
MAX..DURA (MINS): 670.00
RUNOFF VOL.(CU.M.): 5460.13 AT B/WELL ST.
4205.02 AT FOUL SIM
STORM
DISCHARGE (CUMECS)
B/WELL ST.
TIME (MINS)
X10¹

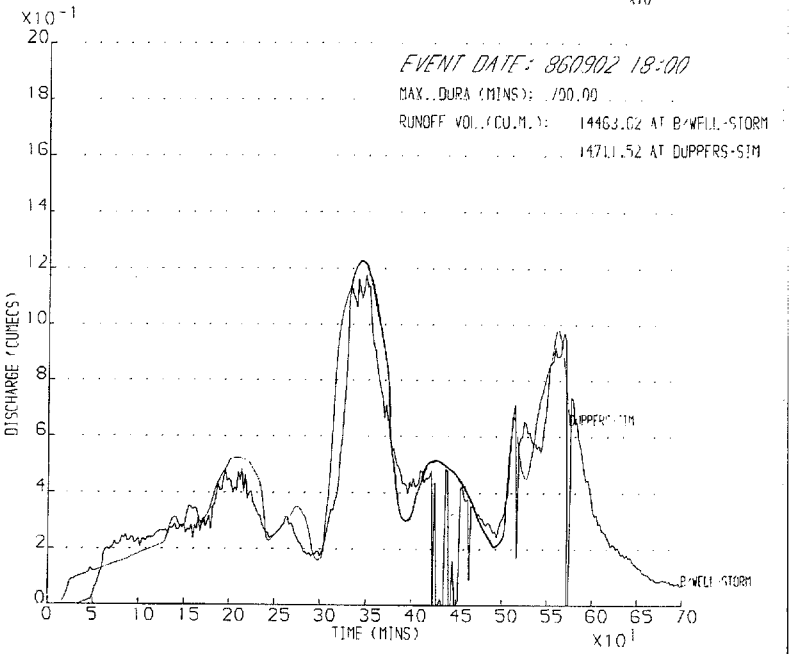

RAINFALL (MM/HR)
OLD KIRK PLACE
TOTAL RAIN (MM): 14.20
X10²
X10⁻²
EVENT DATE: 860815 01:00
MAX. DURA (MINS): 1450.00
RUNOFF VOL. (CU.M.): 10986.70 AT B'WELL-FOUL
7225.25 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
B'WELL-FOUL
TIME (MINS)
X10²

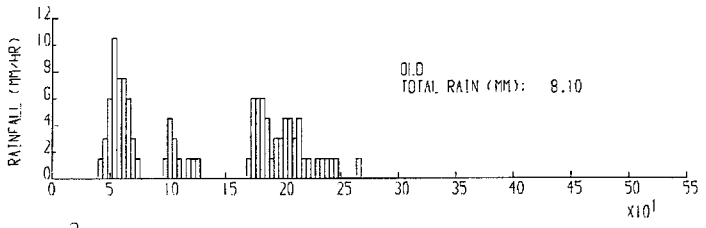

RAINFALL (MM/HR)
OLD KIRK PLACE
TOTAL RAIN (MM): 14.20
X10²
X10⁻¹
EVENT DATE: 860815 01:00
MAX. DURA (MINS): 1450.00
RUNOFF VOL. (CU.M.): 9744.36 AT B'WELL-STORM
7592.36 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
B'WELL-STORM
TIME (MINS)
X10²

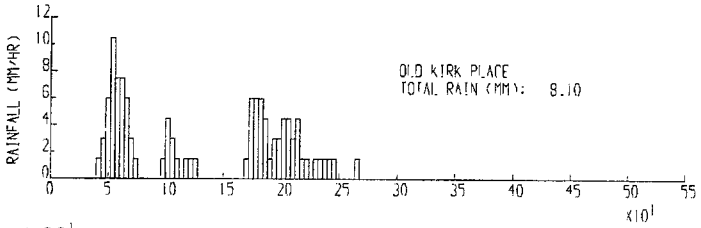

BRUCEFIELD HOUSE
TOTAL RAIN (MM): 18.10
RAINFALL (MM/HR)
X10¹

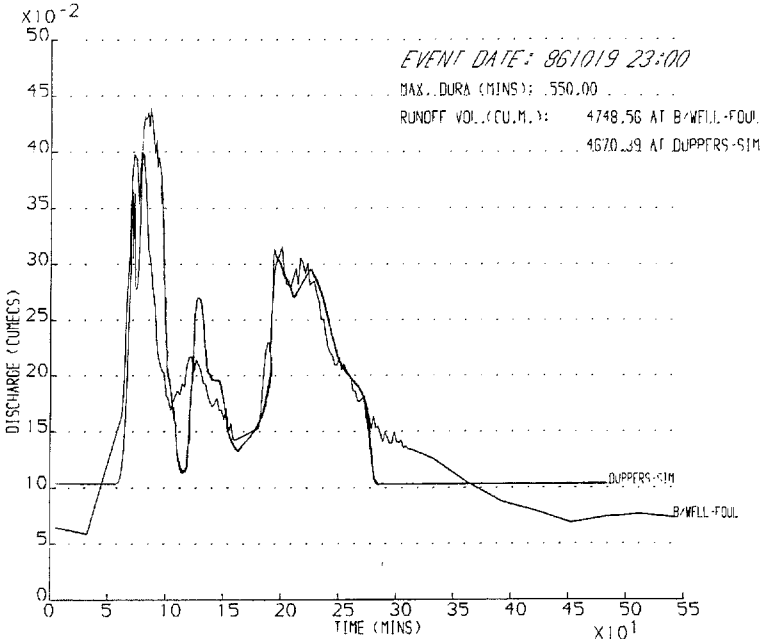

X10⁻²
EVENT DATE: 860902 18:00
MAX..DURA (MINS): 700.00
RUNOFF VOL.(CU.M.): 6249.02 AT B'WELL-FOUL
5946.60 AT DUPPERS-SIM
DISCHARGE (CUMECS)
B'WELL-FOUL
TIME (MINS)
X10¹

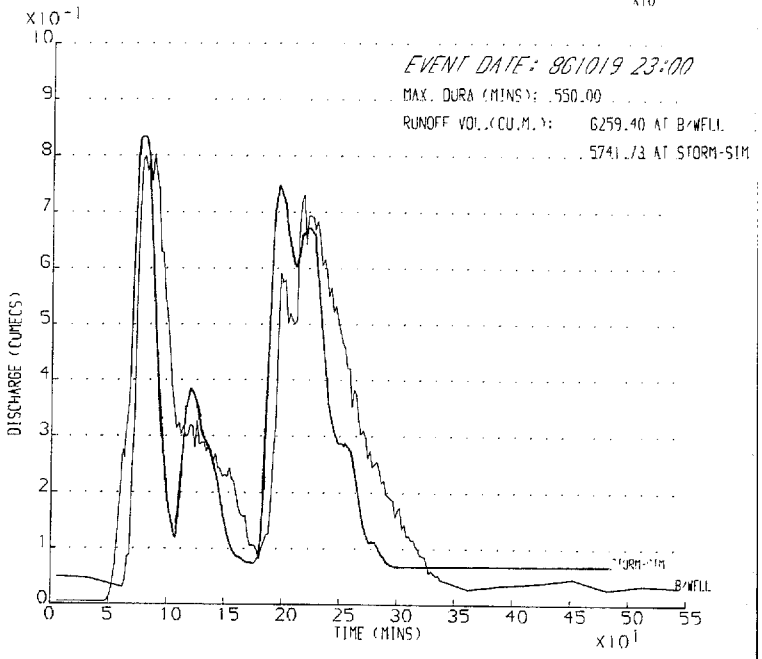

BRUCEFIELD HOUSE
TOTAL RAIN (MM): 18.10
RAINFALL (MM/HR)
X10¹


X10⁻¹
EVENT DATE: 860902 18:00
MAX..DURA (MINS): 700.00
RUNOFF VOL.(CU.M.): 14463.02 AT B'WELL-STORM
14711.52 AT DUPPERS-SIM
DISCHARGE (CUMECS)
B'WELL-STORM
TIME (MINS)
X10¹


RAINFALL (MM/HR)
OLD
TOTAL RAIN (MM): 8.10
X10¹


RAINFALL (MM/HR)
OLD KIRK PLACE
TOTAL RAIN (MM): 8.10
X10¹


X10⁻²
EVENT DATE: 861019 23:00
MAX. DURA (MINS): 550.00
RUNOFF VOL.(CU.M.): 4748.56 AT B/WFLL-FOUL
4670.39 AT DUPPERS-SIM
DISCHARGE (CUMECS)
DUPPERS-SIM
B/WFLL-FOUL
TIME (MINS)
X10¹

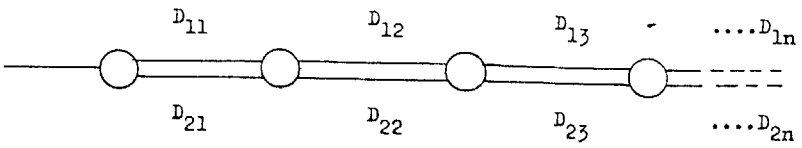

X10⁻¹
EVENT DATE: 861019 23:00
MAX. DURA (MINS): 550.00
RUNOFF VOL.(CU.M.): 6259.40 AT B/WFLL
5741.73 AT STORM-SIM
DISCHARGE (CUMECS)
B/WFLL
TIME (MINS)
X10¹

APPENDIX D

METHODOLOGY OF COMBINING PARALLEL PIPES INTO A SINGLE EQUIVALENT PIPE DIAMETER

APPENDIX D1

PRELIMINARY SIMULATION OF PARALLEL PIPES - LYNE BURN SEWER



D_{1n} , D_{2n} - Diameters of adjacent pipes

L_{1n} , L_{2n} - Lengths of adjacent pipes

Theory The friction head loss h across any two parallel pipes is constant for a given flow rate Q_{1n} & Q_{2n}:

$$h_{1n} = h_{2n} \;:-\; K_{1n}Q_{1n}^{2} = K_{2n}Q_{2n}^{2} \quad \ldots (1)$$

where $K_{in} = \dfrac{f_{in}L_{in}}{3\,D_{in}^{5}}$

putting $h_{1n} = h_{2n} = K_{en}(Q_{1n} + Q_{2n})^{2}$

$$\text{then } \frac{1}{K_{en}^{\frac{1}{2}}} = \frac{1}{K_{1n}^{\frac{1}{2}}} + \frac{1}{K_{2n}^{\frac{1}{2}}} \quad \ldots (2)$$

K_{en} is the equivalent hydraulic resistance of the parallel pipe. Taking the length of the equivalent pipe as equal to the greatest of the parallel pipe lengths L_{1n} or L_{2n}, then the equivalent pipe diameter D_{en} may be determined using eqn (1) , (2) and $K_{en} = \dfrac{f_{en}L_{en}}{3\,D_{en}^{5}}$.

This is unfortunately complicated by the unknown Darcy friction factors f_{in}, which cannot be dtermined without knowing Reynolds Number, or the flow Q_{in}, as illustrated from the attached Moody Diagram. To overcome this it should be assumed that flow must be in the rough turbulent region, and hence f is independent of Reynolds number. The next page gives an example :-

Example of estimation of equivalent single pipe size to represent parallel pipe Lyne Burn sewer.

Data :- Pipe 1 600mm diameter, length 65m

Pipe 2 450mm diameter, length 65m

Both pipes old brickwork - take k_s as 15mm

$$K_{1n} = \frac{f_{1n} L_{1n}}{3 D_{1n}^5} \quad :- \quad \frac{k_s}{D_{1n}} = \frac{15}{600} = 0.025 \text{ from Moody Diagram } f_{1n} = 0.013$$

$$\text{hence } K_{1n} = \frac{0.013 \times 65}{3 \times 0.6^5} = 3.622$$

Similarly $f_{2n} = 0.015$ and $K_{2n} = 17.612$

$$\text{giving } \frac{1}{K_{en}^{\frac{1}{2}}} = \frac{1}{3.622^{\frac{1}{2}}} + \frac{1}{17.612^{\frac{1}{2}}}$$

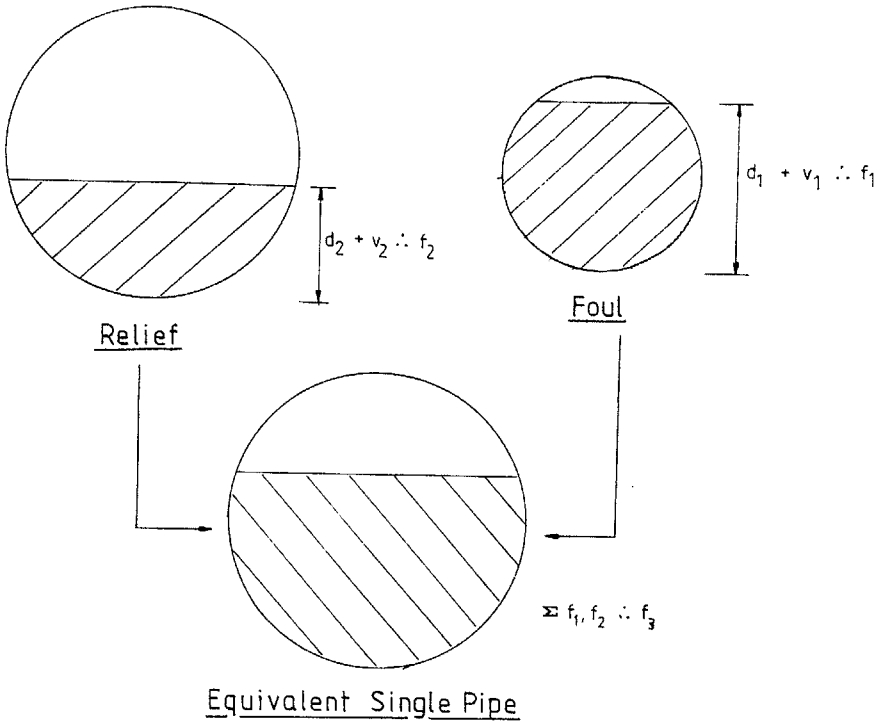
$$\text{and } K_{en} = 1.714 = \frac{f_{en} \times 65}{3 \times D_{en}^5}$$

$$\text{taking } f_{en} = \tfrac{1}{2}(0.013 + 0.015) = 0.014$$

$$\text{gives } D_{en} = \sqrt[5]{\frac{0.014 \times 65}{3 \times 1.714}} = \underline{707 \text{ mm}}$$

Note that the levels at the head and foot of the equivalent pipe should be taken as the lowest of those for either of the actual parallel pipes.

APPENDIX D2



THEORY

From the recorded data i.e. d_1 ,v_1 for the foul sewer and d_2 ,v_2 for the relief sewer, the corresponding flow rate for each sewer could then be determined.

By adding the two flows together during a high flow, the combined flow rate is therefore known,

$$\text{i.e.} \quad f_1 + f_2 = f_3$$

Taking the slope as the average of the two sewers, and using a roughness value of ks equals to 6mm, a combined equivalent pipe diameter can hence be decided from the HRS Tables and Charts.

EXAMPLE

Foul sewer : diameter = 375 mm
slope = 0.82 %
max. capacity = 0.128 m^3/s (at full bore condition)
max. velocity = 1.16 m/s (at full bore condition)

Relief sewer : diameter = 1050 mm
slope = 0.9 %
max. capacity = 2.095 m^3/s (at full bore condition)
max. velocity = 2.42 m/s (at full bore condition)

During an event, the recorded peak flow for foul sewer is :
logged depth = 0.375 m (surcharged)
logged velocity = 1.74 m/s
hence the corresponding flowrate = 0.1922 m^3/s.

Similarly for the storm relief sewer, the recorded peak flow is :
logged depth = 0.394 m
logged velocity = 2.67 m/s
hence the corresponding flowrate = 0.793 m^3/s.

Therefore the total flow rate is 0.9852 m^3/s.

From the HRS Tables, the followings are noted :

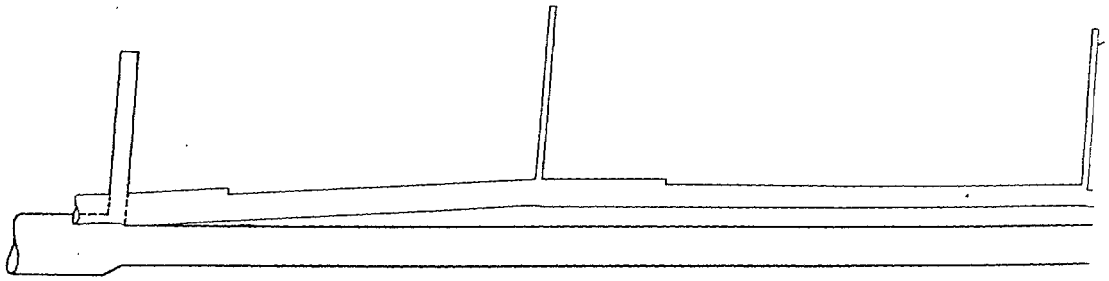
pipe diameter	max. flow rate
800 mm	0.988 m^3/s
825 mm	1.072 m^3/s
900 mm	1.352 m^3/s

Allowing for further surcharge, 900 mm equivalent pipe diameter is therefore used for the computer simulation models.

APPENDIX E

LONGITUDINAL SECTION OF LYNEBURN PARALLEL SEWERS

WITH GEOMETRIC CHARACTERISTICS



DISTANCE (m)

2000 1900 1800 1700

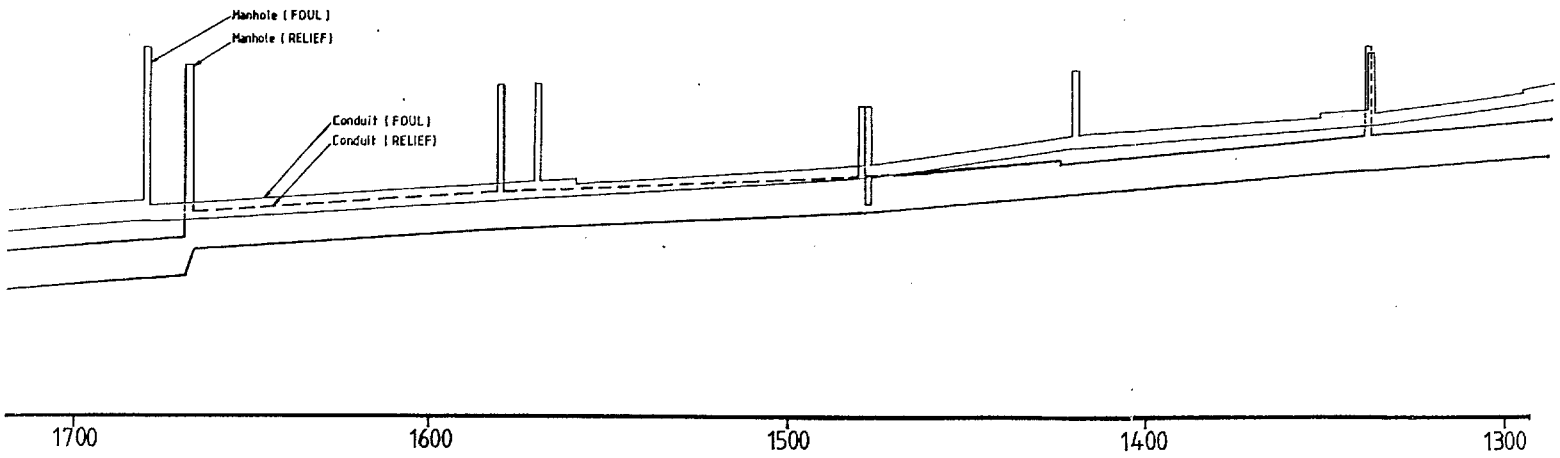
FOUL

Manhole Number			Bathwell Street Chamber (Cross Connection 6)		20		4719		19		4720
Conduit Number			896		989		744		410		
*Capacity	(l/s)		1.2		1.2		0.7		0.7		
Slope	(%)		600x900(Ovoid)		750		750		600		
Diameter	(mm)		28		85		35		115		
Length	(m)										

RELIEF

Manhole Number											
Conduit Number											
*Capacity	(l/s)						18				
Slope	(%)						2388				
Diameter	(mm)						0.7				
Length	(m)						1140				
							277				

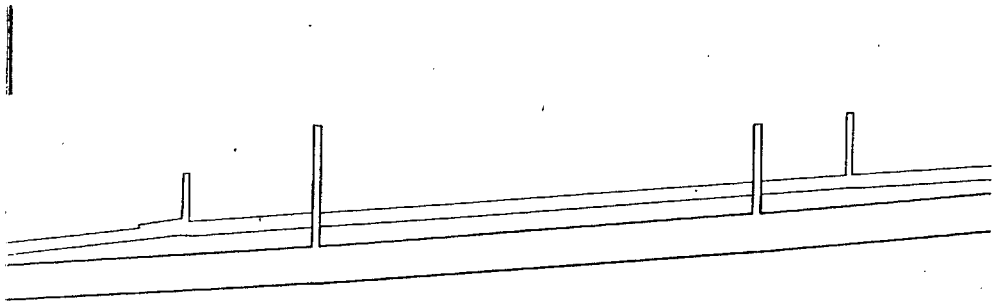
*For a Roughness Value of K_s = 6mm

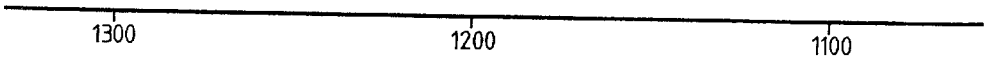


1700 1600 1500 1400 1300

4706		4702		4811 (Cross Connection 5)		4813		4809 (Cross Connection 4)	
18		17		16		15		14	
166		187	115	164		125	204	158	
0.5		0.7	0.7	1.4		0.8	0.8	1.3	
450		450	375	375		375	450	375	
108		10	80	56		67	13	41	

4705		4701		4812		4810
17		16		15		
2223		1850	2487		1991	
0.6		0.5	0.8		0.8	
1140		1140	1140		1050	
86		98	55		84	





ion 4.1

14
158
1.3
375
41

4808
258
1.3
450
12

13
128
0.8
375
184

4805

14
1847
0.7
1050
91

4807

13
2095
0.9
1050
121

4806

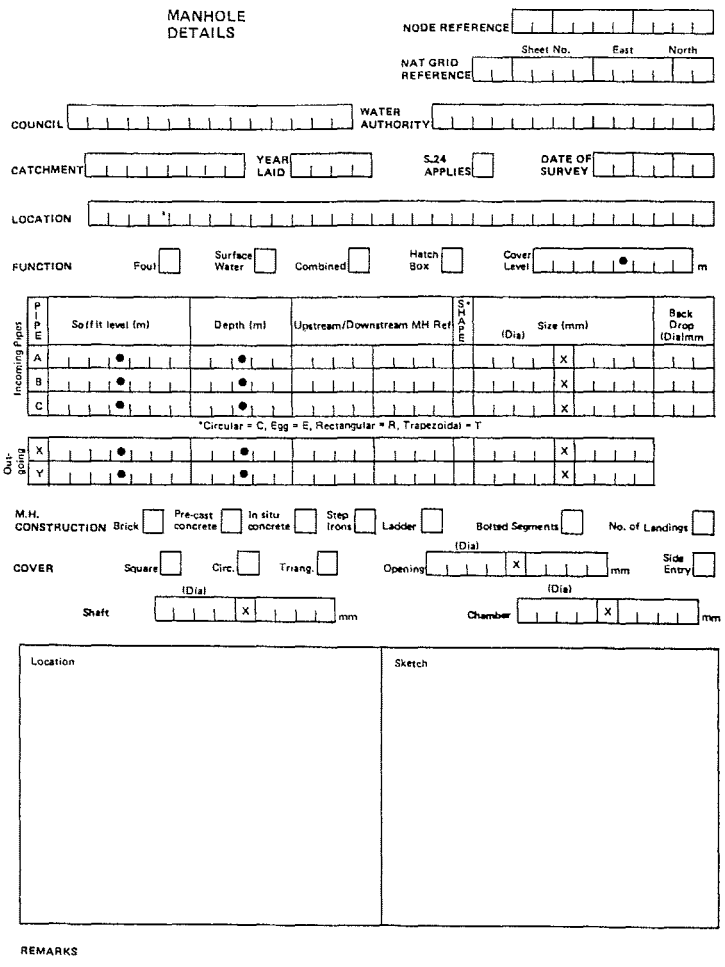
12
2192
1.0
1050
106

APPENDIX F

STANDARDS BOOKING FORMS FOR SEWER SURVEY

AND FOR THE

EQUIPMENT RECORDS IN THE LYNEBURN DRAINAGE SYSTEM STUDY


MANHOLE DETAILS
NODE REFERENCE
Sheet No.
East
North
NAT GRID REFERENCE
COUNCIL
WATER AUTHORITY
CATCHMENT
YEAR LAID
S.24 APPLIES
DATE OF SURVEY
LOCATION
FUNCTION
Foul
Surface Water
Combined
Hatch Box
Cover Level
m
PIPE
Soffit level (m)
Depth (m)
Upstream/Downstream MH Ref.
S*
Size (mm)
(Dia)
Back Drop (Dia)mm
Incoming Pipes
A
B
C
*Circular = C, Egg = E, Rectangular = R, Trapezoidal = T
Out-going
X
Y
M.H. CONSTRUCTION
Brick
Pre-cast concrete
In situ concrete
Step Irons
Ladder
Bolted Segments
No. of Landings
COVER
Square
Circ.
Triang.
Opening
(Dia)
mm
Side Entry
Shaft
Chamber
Location
Sketch
REMARKS

MANHOLE LISTING

MANHOLE NUMBER[1]	FUNCTION OF MANHOLE[2]	DATE			LOCATION OF MANHOLE
		MEASUREMENT		Inspection Details	
		Surveyed	No Assess/ Not Found		

NOTES : (1) The first two digits are the PLAN number, the second two digits are the MANHOLE number shown on that plan.
(2) Surface water (abb. as SW), Foul (F), Foul with overflow (F+O), Combined (C).

0.5-mm (IOH) TIPPING-BUCKET

RAINGAUGE BOOKING FORM

R/G SITE :

NAT. GRID REF. :

DATE OF INSTALLATION :

DAY OF VISIT	DAY No.	DATE	GAUGE READING (NO. OF TIPS)	CORRESPONDING RAINFALL (MM)	REMARKS

EVENT: _____

Started From _____ To _____ (A Total of _____ Mins)

File Type	Site	Data File Duration	Data File Stored in VAX / Mainframe		Catchment Data File	Catchment Data File	SIMULATED OUTPUT FILE	
			Individual	Combined	For running DUCTS	For running WASSP	DUCTS - SIM	WASSP - SIM

REMARKS

PERCENTAGE RUNOFF ESTIMATION

EVENT:_____ DURATION:_____

Site	Contributing Area (Ha)	Rainfall (mm)	Total Runoff (m^3)	Baseflow (m^3)	Net Runoff (m^3)	Percentage Runoff (%)	PEAK FLOW (m^3/s)		
							OBSERVED	DUCTS	WASSP

REMARKS

APPENDIX G

(i) THE LYNEBURN CASE STUDY

BY
W. K. AU YEUNG
C. JEFFERIES
R. M. ASHLEY

Paper presented to the Wallingford Procedure Users Group meeting (WaPUG) by the author at the Council Chambers, Glasgow on 6 November 1986.

(ii) THE DEVELOPMENT OF FLOW SIMULATION MODELS FOR URBAN CATCHMENTS IN NORTH EAST SCOTLAND

BY
W. K. AU YEUNG
R. M. ASHLEY
M. GOODISON
C. JEFFERIES

Paper presented to the HYDROCOMP '89, the International Conference on Interaction of Computational Methods and Measurements in Hydraulics and Hydrology, held in Dubrovnik, Yugoslavia, 13-16 June 1989.

THE LYNEBURN CASE STUDY

W.K. Au Yeung, C. Jefferies, R.M. Ashley
Dundee College of Technology

Introduction

The Lyneburn Sewer in Dunfermline consists of parallel pipes which are interconnected in a more or less random fashion. In all the total length of the parallel pipes is 6.4km with a catchment area of 675ha. The system was described at the WaPUG meeting in autumn of 1985 (Ref.1). In addition to the problem of parallel pipes, another feature of this case study which is of particular interest is the extensive use of the sewered sub-area model.

The study of the Lyneburn Sewer involving Dundee College of Technology (DCT) began in the autumn of 1984. Since then, a programme of underground sewer surveys has been carried out in order to investigate thoroughly the existing drainage system. Firstly, underground manhole and sewer surveys were carried out so that hidden details could be found and also, to a large extent, to update the existing but confusing drainage information - especially the locations of the overflows, interconnections and their associated details. Flows in the system were monitored continuously from September 1985 until October 1986 and were 'sequential' in manner, i.e. two loggers were installed throughout the 13 months on both inflow pipes of the main overflow chamber at Bothwell Street (which was selected to be the lower end of the system for the purposes of the study) and another 4 loggers were located at selected sites. The latter were moved after observation of a number of events, regard being given to the significance of the sub-catchments being monitored at each location. Figure 1 shows the locations and Table 1 gives the sites, sewer type and the duration of each installation.

Sewered Sub-Area Model

The study catchment is complex and Micro-WASSP has limited memory capacity, so successful simulation of the whole catchment has depended largely on simplification of the network. In order to achieve a workable computer model, the Sewered Sub-Area Model (SSAM) has been used widely (in all 48 contributing subcatchments have been simplified in this way). Overland surveys were carried out in detail on two self-contained sub-catchments which were considered to be typical of the catchment as a whole. A detailed simulation of the two study sub-catchments using both WASSP-SIM (full model) and WASSP-SSAM were carried out initially. The predicted output hydrographs showed a close fit with the observed data and the SSAM output proved to be close to both the observed and WASSP full model hydrographs. The topographical parameters derived in this way were then applied to the other areas so that each could be represented by a SSAM. This meant that the total number of pipes could be reduced below the Micro-WASSP limit of 300.

Parallel Pipes

Besides the main trunk Lyneburn Sewer, the secondary branches of the network including the Calaisburn Park and Bellyeoman Sewers are all parallel pipe systems (Fig. 1 shows their positions). A major aim of this study is to develop a model which will allow simulation of flows in the individual parallel sewers. In order to achieve this the model must be able to represent storm overflows and interconnecting pipes which allow flows both from the foul to the relief sewers and also, when surcharging occurs, cope with flows in the reverse direction. Neither Micro nor mainframe WASSP is capable of modelling this situation (Ref.2) and simplications must be accepted. To produce this initial approximate model of the parallel pipe system, there appeared to be three possible alternatives:

(i) Adoption of available features in WASSP e.g. storm overflows or off-line tanks;

(ii) Consideration of the network as 2 separate systems which combine at a downstream outfall;

(iii) Combination into one single equivalent pipe system.

Option (i) was considered to be the better approach but after comparing the actual number of interconnections and the permissible limit of only ten storage tanks in Micro-WASSP, this approach had to be abandoned. The second option would simply exceed the allowable number of pipes when the full SSD file was totally assembled and option (iii) was therefore used. The simplification of the twin sewers into one was carried out using the Darcy equation on the assumption that the Reynolds Number would be high enough for flow to be in the turbulent zone and the friction gradient between manholes the same in each pipe. The actual pipe lengths and the number of intermediate manholes on the equivalent pipes were retained as being the same as for the main and secondary sewers when considered separately.

Model Verification

Simulation of the catchment as a whole showed good-fit with the observed data based on the combined foul and relief flows. However, the fit between the observed and predicted results started deviating as more extreme events were used i.e. rainfall of approximately 20mm or above. Despite revision of the contributing areas in the SSD data, some 20-30 percent of flow was still not accounted for when compared with observed hydrographs. This pointed to the possibility of some extra as yet 'hidden' contributing area somewhere in the catchment. A better simulation was obtained after a Percentage Runoff was input in the PCD file rather than using SOIL and UCWI. This figure was developed using the graph of Runoff Volume (RUNVOL) vs. Rainfall (Fig. 2). The output confirmed that there were approximately 10-15ha of contributing area missing. This discrepancy was resolved by including two remote sub-catchments at the head of the system to the east of Dunfermline which, in the initial specification of the catchment were assumed only to contribute dry weather flow to the Lyneburn Sewer. The storm flow from these areas normally finds its way to the nearby Mowbray Burn but during severe events, flow level rises and starts contributing to the sewer system. These extra areas have since been included in the data and a closer comparison obtained.

Limitations of WASSP

Following the building and verification of the primary model for this catchment, the second stage of this work will be to enhance the model in order to route the flows more precisely to give the runoff hydrographs for each of the individual parallel Lyneburn Sewers. Since WASSP is not

able to model parallel pipe systems directly, further simulators will be considered. The US SWMM model (Ref. 3) is being investigated and DUCTS (short for Dundee College of Technology Sewerage Simulation) (Ref. 4), which is based on published reports (Ref. 5), is currently under modification to a twin pipe model. For models of this nature to operate correctly, it is important that precise details of the cross connections, sizes and levels be ascertained so that any reverse flows caused by surcharging can be accounted for as flow is routed through the system.

The study has shown that model construction requires a lot of revision and enhancing of SSD files to obtain good fits. One batch of events is normally recommended to verify the model and another to check its operation. Although relatively simple to use, the WASSP package does suffer from minor deficiencies such as an inability to take very large systems (Micro-WASSP) and long duration events and there is a maximum simulation time of only 480 minutes on Micro-WASSP. In addition, WASSP cannot model satisfactorily the type of parallel pipe problem described. The sewered sub-area model has given close results when compared with the full model and could be adopted on most sewer networks, especially where system simplification is required.

References

1. Ashley, R.M., Jefferies, C., Au Yeung, W.K. Minimum Flow Survey Networks, WaPUG Autumn Meeting, Birmingham City Council, 1985.
2. Chaplin, R. Experience with storm overflows, WaPUG Autumn Meeting, Birmingham City Council, 1985.
3. Lagger, J.A., Shubinski, R.P. & Russell, L.W. Development of a Simulation Model for Stormwater Management. J.Wat.Poll.Conr.Fed., 43 p2425-35. (1971)
4. Ashley, R.M., Jefferies, C. DUCTS - Application of the Computer Simulation Model to Dunfermline, Dundee College of Technology, 1984.
5. Various authors. Studies in Urban Hydrology - A Summary of Urban Hydrology Research 1974 - 1979, Institute of Hydrology, December 1980.

NO	SITE NAME	SEWER TYPE	DURATION
1	Bothwell Street Chamber	Foul	30/9/85 to 13/10/86
7	Bothwell Street Chamber	Relief	9/12/85 to 13/10/86
2	St. John's Drive	Relief	30/9/85 to 3/12/85
4	Rex Park	Relief	30/9/85 to 31/7/86
5	Rex Park	Foul	30/9/85 to 20/6/86
6	Wallace Street	Relief	31/7/86 to 1/9/86
10	Wallace Street	Foul	31/7/86 to 1/9/86
8	Mill Road	Foul	13/6/86 to 29/9/86
9	Mill Road	Relief	16/7/86 to 29/9/86
11	Erskine/Beveridge Court	Relief	1/9/86 to 29/9/86
12	Erskine/Beveridge Court	Combined	1/9/86 to 13/10/86
13	Millhill Street	Relief	29/9/86 to 13/10/86
14	Millhill Street	Foul	29/9/86 to 13/10/86

Table 1 - Logger locations and duration of monitoring.

FIG. 2 - RUNVOL VS. RAINFALL

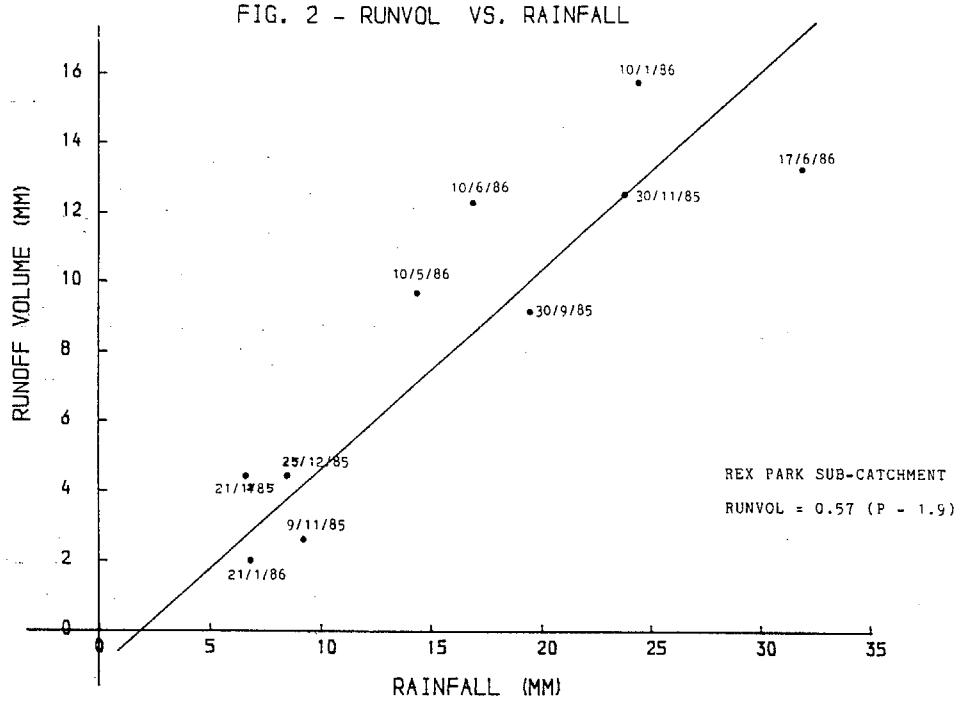
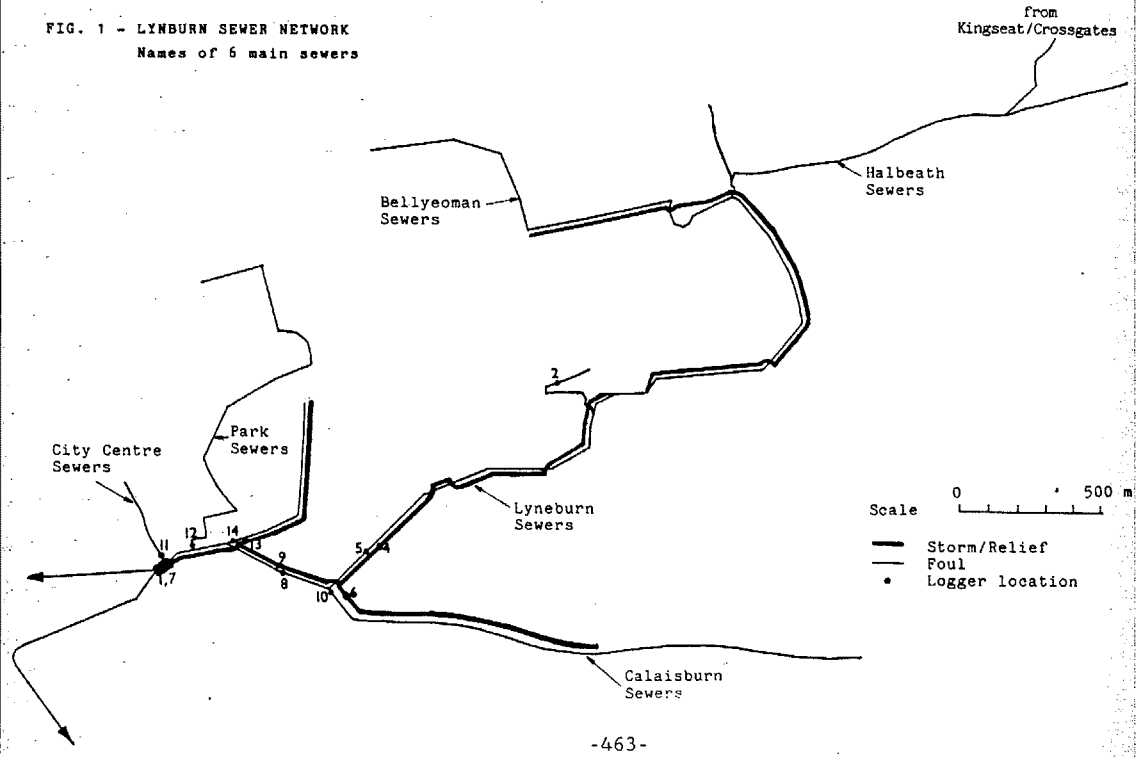


FIG. 1 - LYNBURN SEWER NETWORK
Names of 6 main sewers



THE DEVELOPMENT OF FLOW SIMULATION MODELS FOR URBAN CATCHMENTS IN NORTH EAST SCOTLAND

W.K.Au Yeung, R.M.Ashley, M.Goodison & C.Jefferies

Department of civil engineering, surveying & building
Dundee Institute of Technology
Bell Street, Dundee, DD1 1HG, Scotland

ABSTRACT

Sewer flow simulation modelling for flood alleviation and pollution control purposes for urban catchments in north-eastern Scotland varying in size up to about 30 sq.km has highlighted the difficulties of applying National standard models based on limited data bases. The models developed are described and the actual catchment data recorded and used in their development is considered in terms of the lessons learned for future model development.

KEYWORDS

Data collection, sewer flow monitors and measurements, overland flow, simulation models, calibration, parallel pipe computer model.

INTRODUCTION

Extensive programmes coordinated by staff of Dundee Institute of Technology for the collection of hydrological data for the purpose of setting up simulation models for surface water and sewered drainage systems have been underway since 1982 as described by Ashley & Jefferies (1989). The programmes have been undertaken collaboratively with municipal authorities in north-eastern Scotland and cover the two main cities of Dundee (population 120,000) and Dunfermline (population 60,000 and subject to considerable urban expansion). The original aim was to develop simulation models for the combined sewerage systems which could be used as operational management tools to avoid system overload and surface flooding. More recently the scope of the work has been enlarged to include water quality aspects as well as hydraulic factors in response to the growing recognition in the UK of the importance of controlling in-sewer sedimentation and pollution of watercourses from outfalls and overflows.

Initial studies concentrated on the natural watercourse catchment in Dunfermline and the use of the UK Flood Studies Report methods for flood estimation were described by Jefferies et al (1986). Since then the US HEC II surface profile software has also been applied to the main watercourse. Subsequently the studies were extended to include the combined sewerage system using a strategy detailed by Ashley et al (1986), and for which UK WASSP models (National Water Council, (1982) were initially developed based on standard methodologies as described by Price & Osborne (1986), but with the use of actual catchment rainfall-runoff data, model calibration rather than simply verification was found to be necessary. Due to the complexity of the sewerage network, which comprises a central core of 6.4km of parallel interconnected sewers, the standard WASSP model was found to be inadequate for modelling the flows in the individual branches of the parallel sewers. A unique sewer flow simulator (acronym DUCTS) has since been developed specifically to model the hydraulics of the parallel sewers.

The combined sewer system in Dundee is also unusual as it has more then 300 hinged-door 'control gates' which can be used to divert flows through a bewildering variety of different routes. Current application of the WASSP and recently released WALLRUS simulation models to the system has again confirmed the difficulties in using standard models for complex sewer systems. The problems of using standard models together with an appraisal of the methodologies currently adopted by the UK water industry for model verification have previously been discussed by Ashley & Jefferies (1987). Considerable experience has been gained in sewer flow monitoring at some 100 locations in north-east Scotland, some of which have been monitored for more than a year, and the data collected and models developed therefrom has confirmed the necessity to calibrate models using real catchment data.

IN-SEWER FLOW LOGGING

Flow surveys

The primary rainfall and corresponding in-sewer flow data have been collected using standard high resolution raingauges and intrusive in-sewer flow survey units as supplied by Detectronic Ltd. Short-term sequential flow surveys have been carried out using a small number of units moved around catchments, rather than the saturation flow surveys of short duration recommended by Price & Osborne (1986) for verification of models.

Flow loggers

The flow loggers used operate using a pressure transducer to measure depth and ultrasonic transducers to measure average velocity. The relative precision of these instruments has been considered by Ashley et al (1986) and more recently by Burrows et al (1989). The Flow Survey Manual published by the UK Water research centre (1987) also sets out the criteria for their use and recommends procedures to ensure that the transducers are functioning correctly. With good hydraulic conditions these ultrasonic units may be relied upon to provide data which is within 20% of the actual mean discharge at a given site. On-site calibration can improve this figure to perhaps 10%, but not normally over the whole range of flow conditions.

Potentially the most accurate technique for velocity measurement is to use electromagnetic (EM) systems. Unfortunately the intrusive type of EM as produced in the USA has been found to be unreliable for in-sewer work. The primary problem is that of greasing, which causes insulation between

the EM sensor head and the flow leading to inaccuracy in readings in combined sewers. The difficulties posed by greasing can only be overcome by daily cleaning of the velocity sensor.

The ultrasonic type of unit measures velocity over a 'field' (albeit unknown in extent) whereas the EM transducer measures velocity within a small range around the head, typically 25mm in extent. The manufacturers of EM systems such as the Flo-Tote (Marsh-McBurney) claim that by assuming a logarithmic velocity distribution across the depth of the flow the mean velocity and hence discharge can be evaluated from this localised measurement. With the range of conditions encountered in sewer systems any such assumption would appear to be highly dubious for the accurate measurement of flow. The ultrasonic system with its 'averaging' at least provides some measure of the actual velocity variation with depth.

RUNOFF MODEL CALIBRATION

Runoff volume and depression storage

The overland flow models developed by the Institute of Hydrology for the Wallingford Procedure were based on a very limited data set which was not representative of all types of catchment (Pratt (1984)), and the use of site-specific calibration data for WASSP modelling is recommended. This can only be done with the WASSP software by determining a value for percentage runoff for each modelled rainfall event and using this real-catchment information globally for model calibration. No depression storage calibration is possible, however. The latest variant of WASSP is the WALLRUS software which incorporates some minor changes in the overland flow model none of which substantially affect the way in which the rate and volume of runoff are determined. Most significantly however, the user now has the potential to modify the percentage runoff for every contributing pipe area, within certain constraints, and thus a more specific calibrated model may be developed using this software if the user has sufficient data. Both WASSP and WALLRUS models are currently being used for the catchments described. Sufficient data has been collected to make some comparisons with the overland flow model used in WASSP and described by Kidd (1978), and also with the work reported by Pratt (1984) for small catchments.

The default equation used by WASSP to determine percentage runoff PRO is:

$$PRO = 0.829\ PIMP + 25\ SOIL + 0.078\ UCWI - 20.7$$

Where PIMP is the catchment percentage impermeability, SOIL is a soil index and UCWI a measure of the antecedent conditions.

The WASSP software allows the user to input a catchment-specific value for SPR where SPR replaces the catchment data thus:

$$PRO = SPR - 0.078\ UCWI$$

With sufficient rainfall-runoff data the user can evaluate SPR from:

$$SPR = \frac{RUNVOL \times 100}{P \times AREAC}$$

Where RUNVOL is runoff volume, P rainfall volume and AREAC the catchment area.

RUNVOL is determined from actual data by producing a plot of rainfall against observed in-sewer flow of the type shown in Figures 1 and 2 and then using a regression analysis for the relationship:

$$RUNVOL = RC \times (P - DEPSTOG) \times AREA$$

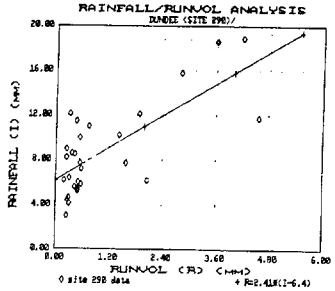


FIGURE 1
DUNDEE MAIN OUTFALL

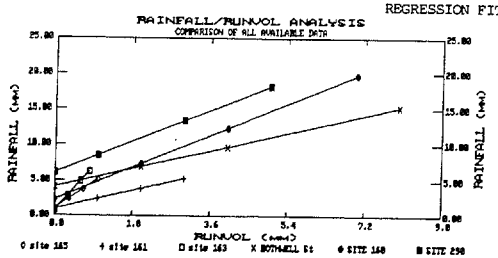


FIGURE 2
REGRESSION FITS FOR DEPSTOG & RC

RC, the runoff coefficient is given from the slope and DEPSTOG, the depression storage from the intercept. There appears to be an inverse correlation of depression storage with the slope of the maximum drainage path length for a catchment, (ibid), and this is illustrated in Figure 3. The Scottish catchments represented in the figures have the characteristics given in Table 1 and are listed in order of increasing catchment size.

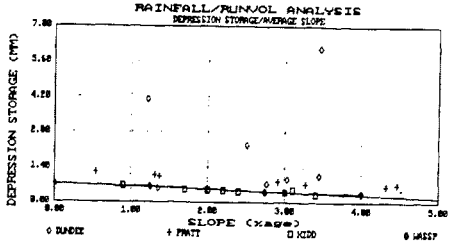


FIGURE 3
DEPRESSION STORAGE - SLOPE OF MAIN
DRAINAGE PATH

TABLE 1 - Catchment details

Catchment Ref	Area (ha)	Length (km)	Slope (%)	RC (%)	DEPSTOG (MM)	Land use
S161	6.69	0.92	3.0	1.44	0.94	Mixed H & I
S165	22.03	1.45	3.5	4.10	1.07	Low density H
S163	43.95	1.84	2.8	6.88	0.76	Med density H
S160	63.77	2.20	2.5	2.50	2.30	Mixed H & I
REX BOTHWELL St	134.18	4.13	1.4	1.75	1.90	City catchment
	319.81	4.88	1.2	1.40	4.10	"
290	592.78	4.18	1.9	2.41	6.14	"

H - Housing, I - Industrial

Data pertaining to the largest catchment reported are shown in Figure 1, where storms covering the period from February 1987 to September 1988 are shown. The plot is comparable with other studies and shows a reasonably consistent value for RC of 2.41 given by the best-fit straight line shown. The depression storage increases with size of catchment for the majority of catchments reported and this is illustrated by the best-fit plots shown in Figure 2. Figure 3 shows the variation of depression storage with main slope compared with the data of Kidd (1978), used for the linear relationship in the WASSP model, and also the data reported by Pratt (1984) for small catchments. Kidd's data covered 14 catchments ranging in size from 0.6 - 247 ha, and Pratt's data were from 10 sites of 0.004 - 0.085 ha. Neither of these included the type of inner city data reported here.

These figures, and those for other catchments, have been used to develop calibrated WASSP models as described by Au Yeung et al (1986) and as illustrated in Figure 4 for the parallel pipe catchment. For all but the smallest of catchments the use of calibration data for SPR has improved the modelled results when verifying the setting up of a WASSP model.

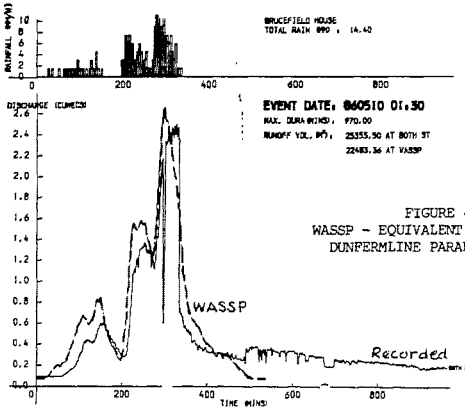


FIGURE 4
WASSP - EQUIVALENT PIPE MODEL RESULTS
DUNFERMLINE PARALLEL PIPE SYSTEM

The smaller of the catchments shown in Table 1 and Figures 2 and 3 are now being modelled using the WALLRUS software. These are the subject of a research programme sponsored by WRc in which the provenance, disposition and movement of sewer sediments are currently being investigated. The initial WASSP model covered all of the catchments combined and was found to be inaccurate when applied to the component sub-catchments due to the inaccuracies of the overland phase of the model. Currently the individual runoff characteristics are being determined for each individual sub-area for use in the more flexible WALLRUS software.

DUNFERMLINE PARALLEL PIPE MODEL

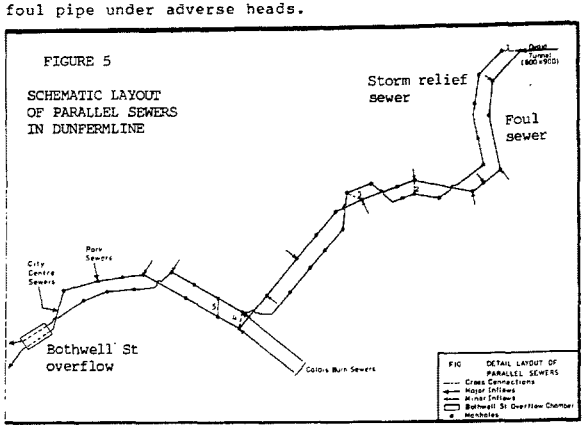
Parallel pipe catchment

The total length of parallel sewers in Dunfermline currently being investigated in detail is 3.2 km long ending at an overflow chamber at Bothwell Street. The system is described by Ashley et al (1986) and Au Yeung et al (1986). The foul and storm relief pipes have five main cross connections at random locations as shown schematically in Figure 5. The largest of the sewers is 2100 mm in diameter and the maximum capacity of the foul and storm relief sewers are 896 l/s and 2,388 l/s respectively. Surface flooding occurs at various points along the parallel pipe system, being particularly severe in the vicinity of the cross-connections. An unusual feature of the

system is the occurrence of reverse flow through the cross-connections from the storm relief sewer back into the foul pipe under adverse heads.

FIGURE 5

SCHEMATIC LAYOUT OF PARALLEL SEWERS IN DUNFERMLINE



System survey work was carried out to determine details of the sewer catchment and also the physical characteristics of the cross-connections. A flow monitoring programme was implemented which included the collection of data for flows and levels which occurred simultaneously in the storm and relief sewers for a range of events. Two loggers were installed at the catchment outfalls for a period of nine months and between five and ten usable events were recorded during this time at each of the other cross-connections.

WASSP equivalent pipe model

The overall model was initially developed by setting up a series of component sub-models as described by Au Yeung et al (1986). The parallel pipe system was first modelled using the WASSP package with the twin pipes combined into one single equivalent length. Combination was based on the assumption that the friction head loss in each parallel pipe was constant overall and that the the two pipes have similar hydraulic resistance characteristics. The overall catchment runoff was used to produce a calibrated model from recorded percentage runoff data. An example of the output from the model is shown in Figure 4 which gives the modelled and observed hydrographs for a rainfall of 14.4 mm. The Figure is typical of the comparison obtained between WASSP modelled and observed hydrographs for the 'lumped' parallel pipe model.

Parallel pipe model

In order to proceed with the investigations required to produce design solutions, the initial model had to be refined to predict the performance of the flows in the individual legs of the parallel sewers. It was known that surcharging occurred in the cross-connection manholes for rainfall as low as 6.8 mm and that overflow occurred with a rainfall intensity of 1.5 mm/h. The worst storm observed had 32.6 mm of rain in a short time and under these conditions the sewer flow loggers showed that a balance of heads existed in the surcharged parallel pipes.

Prior to model development a complete understanding of the performance of the cross-connections was required. The conditions required for overflow, reverse flow, and head balance were determined and worst cases postulated. It was concluded that reverse flow was virtually impossible for four of the five connections, but for the fifth, it commonly occurred under only 220 mm of surcharge in the relief sewer. These investigations enabled a specification to be drawn up for the requirements of a site-specific parallel pipe sewer flow model, DUCTS. These are:

- the hydraulic performance of both the foul and relief sewers should be simulated;
- flow and level hydrographs should be available from the model for any specified location on the parallel sewers;
- the model must be able to simulate the through flow, the overflow, and reverse flow for either of the two types of cross connection;
- major inflows must be accomodated within the model at any point within the parallel pipe system;
- the component sub-models to be used for the rainfall, overland flow, and sewer flow should be based on those used in the Wallingford Procedure and should include the sewered sub-area model used therein.

The non-availability of commercial software that could deal with surcharged overflows or non-dendritic pipe systems meant that a unique model had to be developed for this system. Despite being set up initially to deal with this parallel pipe system, this model DUCTS, is flexible and can be used to model any complex system which has non-standard sewerage. Essentially the limitations of the software are similar to those for WASSP in terms of the component sub-models, except for DUCTS capability to handle unusual features. The DUCTS model is based on the rainfall, runoff and below ground models used in the development of WASSP and as described in the UK urban hydrology studies research reports produced by the Institute of Hydrology (1980), and by Bettess & Price (1978). Modifications to the code can be made to enhance the runoff model to make it more flexible than the WASSP software.
The assumptions and limitations of the parallel pipe model are:

- the length of each bridging pipe may not exceed ten times the pipe diameter;
- bridging pipes are used solely for flow connection purposes and hence no flow routing processes are used in these pipes;
- any direct flows into the cross-connection manholes must be input into the pipe immediately upstream;
- successive cross-connections must be separated by at least one discrete pair of parallel pipes;
- head losses at cross-connection manholes are ignored unless the manholes are surcharged;
- currently a maximum number of ten such cross-connections are allowed.

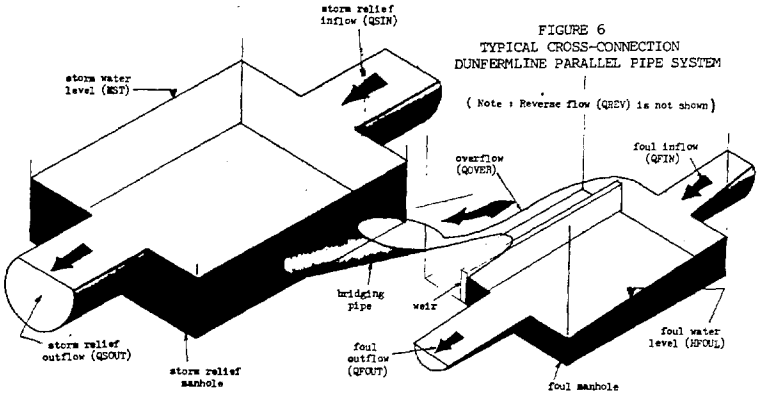
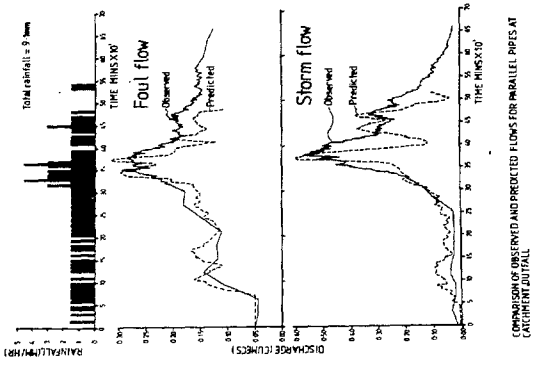


FIGURE 6
TYPICAL CROSS-CONNECTION
DUNFERMLINE PARALLEL PIPE SYSTEM

(Note : Reverse flow (QREV) is not shown)

The flow chart in Figure 7 shows the structure of the parallel pipe model, and Figure 6 the behaviour at a typical cross-connection under free surface conditions. Overflows under non-surcharged conditions are determined using the weir equation with coefficients evaluated from the Rehbock equation. The levels in the foul and relief manholes are compared and the onset of surcharging is detected, and/or the flow direction determined.



COMPARISON OF OBSERVED AND PREDICTED FLOWS FOR PARALLEL PIPES AT CATCHMENT OUTFALL

FIGURE 8
PARALLEL PIPE MODEL -- DUCTS
FLOWS IN INDIVIDUAL SEWER LEGS

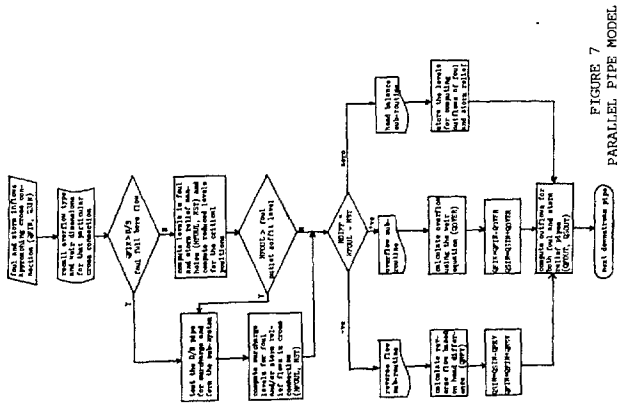


FIGURE 7
PARALLEL PIPE MODEL
FLOW CHART

The DUCTS parallel pipe model has been extensively tested for the Dunfermline catchment. The testing has dealt with the consistency of the flow behaviour in the overall system and particularly at cross-connections, and also the robustness of the model. A single cross-connection with three contributary sub-catchments was subjected to a constant rainfall input of 3 mm/h which was then increased incrementally to 96 mm/h. The distribution of flows within a basic component of the parallel pipe system was thus determined over an extreme range of rainfall conditions. This process was repeated for a pair of cross-connections and then the overall system was modelled with the same range of constant rainfall intensities. Surcharging and reverse flows were also 'forced' by reducing the diameters of downstream pipes thus allowing flow and level consistency to be checked. Finally the model was used to simulate observed events and the output compared with monitored flows and levels. Figure 8 shows a typical comparison between observed and simulated hydrographs for flows in each of the foul and relief sewer branches for a storm of 9.1mm.

CONCLUSIONS

Standard sewer flow simulation models have been developed using limited data bases which may not necessarily apply to all types of catchment or sewerage systems. Verification of individual models using computer software may reveal erroneous system data but may not be precise enough for detailed modelling particularly where water quality aspects are to be considered. Models should be capable of refinement through calibration for site-specific conditions particularly for overland flow processes. The UK WASSP software has been used successfully for flow simulation over the last eight years, despite having only a very limited facility for calibration. The limitations of the overland flow model representation of depression storage and runoff volume are most marked for city centre and larger catchments. The recently released WALLRUS software has potentially more scope for overland flow model calibration and different characteristics may be assigned to individual contributing pipe lengths. New initiatives by Hydraulics Research Ltd and the UK Institute of Hydrology to develop an enhanced overland flow model using a much more extensive data base are expected to result in an improved UK model for use in 1990. This model will be more compatible with the requirements for more precise quality as well as sewer flow modelling.

Current techniques for in-sewer flow logging are reasonably accurate and reliable using robust ultrasonic systems to measure average flow velocity. Trials with electromagnetic systems have shown them to be less reliable and prone to error in reading when inserted in combined sewer systems due to the build-up of a grease layer which insulates the sensor from the flow. Even when operating, this type of system which measures the velocity very close to the sensor head, is of doubtful value for the determination of average velocities and rates of flow as it relies on the assumption of a pre-definable logarithmic velocity distribution. Such assumptions may be realistic in laboratory conditions, but are unlikely to be reasonable in real sewer systems where hydraulic conditions are far from ideal. All flow data recorded in sewer sytems should be viewed with a degree of scepticism as this is likely to be accurate to only within about 10% of the true flow at best, and more typically to a relative accuracy of about 20%.

REFERENCES

Ashley, R.M. Jefferies, C. & Stevens, G.S.W. (1986) "Overloading of the Dunfermline sewerage system due to peripheral urban expansion". Proc ICE Cent. Conf. on Infrastructure renovation and waste control. UMIST UK, April 1986.

Ashley, R.M. Jefferies, C. (1987) "Sewerage simulation -are standard methods appropriate?" Proc. 4th Int Conf on Urban Strom Drainage, Lausanne, Aug 1987.

Ashley, R.M. & Jefferies, C. (1989) "Recent drainage research work in Dundee". Civil Engineering Technology Vol 11, No.3, June 1989.

Au Yeung, W.K. Jefferies, C. & Ashley, R.M. (1986) "The Lyneburn case study". Wallingford Procedure users group meeting, Glasgow Nov 1986.

Bettess, R. & Price, R. (1978) "A mathematical surcharging model for storm sewer systems". Hydraulics Research Station Rep IT 178. June 1978.

Burrows, R. Crowder, G. & Lowther, J. (1989) "Sewer flow surveys: an appraisal of combined velocity/depth logger performance by laboratory studies". Proc Conf. on Drainage & Waste Management into the 1990s. Dundee Inst of Tech. May 1989.

Institute of Hydrology (1980). "Studies in urban hydrology". UK Dec 1980.

Jefferies, C. Stevens, G.S.W. & Ashley, R.M. (1986) "A Scottish Burn". Journal IWES, UK, Dec 86, Vol 40, No.6.

Kidd, C.H.R. (1978) "A calibrated model for the simulation of the inlet hydrograph for fully sewered catchments". Proc. Int Conf on Urban storm drainage, Southampton, UK, April 1978.

National Water Council (UK) (1982) "The Wallingford Procedure for the design and analysis of urban storm drainage".

Pratt, C.J. (1984) "Wallingford Procedure - hydrological sub-models: their development and application". Proc Seminar on the Wallingford Procedure, Dundee College of Tech. June 1984.

Price, R.K. & Osborne, M.P. (1986) "Verification of sewer simulation models" Proc. Int Symp UDM'86. Dubrovnik April 1986.

Water Research centre (UK) (1987) "A guide to short term flow surveys of sewer systems".