Appendix 1

** SILOCA OUTPUT **

LOCALIZACAO EM PLANTA DOS NOS DO SISTEMA
(nodes location)

NO	LOC	NO	LOC	NO	LOC	NO	LOC	NO	LOC
1	A2	5	C4	9	B3	13	A3		
2	A3	6	C4	10	B3	14	B2		
3	B1	7	C2	11	A4	15	C1		
4	A1	8	B2	12	B2	16	A3		

LOCALIZACAO EM PLANTA DOS TROCOS DO SISTEMA
(pipes location)

TROCO	LOCALIZACAO		TROCO	LOCALIZACAO		TROCO	LOCALIZACAO	
1	A2	A3	8	C2	B2	15	B2	A3
2	A3	B1	9	B2	B1	16	B2	A3
3	B1	A1	10	C4	B3	17	A3	B2
4	A2	A3	11	B3		18	B1	B2
5	A3	C4	12	B3	A4	19	B2	C1
6	C4		13	A4	C2	20	C1	A3
7	C4	C2	14	C2	B2	21	A1	A3

** SIMALI OUTPUT **

```
*********************************************************************
*                                                                   *
* LABORATORIO NACIONAL DE ENGENHARIA CIVIL - DH/NHS                 *
*                                                                   *
* CALCULO HIDRAULICO DE SISTEMAS DE DISTRIBUICAO DE AGUA EM PRESSAO *
*                                                                   *
* SIMALI - VERSAO DE 1986                                           *
*                                                                   *
*********************************************************************
```

TITULO: Z. B. DE MATAQUENE

DATA: NOV 1986

FOI UTILIZADA A EXPRESSAO DE HAZEN-WILLIAMS

AS ITERACOES TERMINAM QUANDO O MAX. DELTA Q <=0.001 l/s (tolerance)

O FACTOR MULTIPLICATIVO DOS CONSUMOS E: 1.000 (demand multiplying factor)

O SISTEMA APRESENTA 21 TROCOS E 16 NOS (number of pipes and nodes)

AS CARACTERISTICAS GEOMETRICAS DOS TROCOS SAO: (network configuration)

pipe	nodes number		lenght	dia.	Hazen-Will.	pipe use coeff.	local head loss coeff.
TROCO	NO DE MONT.	NO DE JUS.	COMPRIM. (m)	DIAMETRO (mm)	COEF.H-W	COEF.DE UTILIZ.	COEF. DE FCL
1	1	2	100.0	300.0	140.0	1.000	0.00E+00
2	2	3	770.0	125.0	0.0	1.500	0.00E+00
3	3	4	500.0	150.0	0.0	0.550	0.00E+00
...			...		...		
20	15	16	320.0	125.0	0.0	1.000	0.00E+00
21	4	16	30.0	300.0	0.0	0.000	0.00E+00

O CONSUMO TOTAL DO SISTEMA E DE: 141.05 l/s (total consumption)

AS CARACTERISTICAS DOS RESERVATORIOS, CENTRAIS ELEVATORIAS
OU OUTROS NOS COM CAUDAIS VARIAVEIS SAO: (reservoirs and pumps)

NO	N. DE BOMBAS EM PARALELO	PARAMETROS P1	P2	P3	P4	COTA (m)
11	1.	10.000E+00	0.000E+00	0.000E+00	0.000E+00	40.5

NO. DE MALHAS = 6 (loops)

NO. DE ITERACOES = 8 (iterations)

AS CARACTERISTICAS HIDRAULICAS NOS TROCOS SAO: (hydraulic results)

pipe	lenght	dia.	flow	tot.head loss	head loss	veloc.
TROCO	COMPRIM. (m)	DIAMET. (mm)	CAUDAL (l/s)	PER.CAR.TOT. (m)	PER.CAR.UNIT. (m/km)	VELOCID. (m/s)
1	100.0	300.0	105.44	1.114	6.19	1.49
2	770.0	125.0	7.61	17.525	22.76	0.62
4	1050.0	300.0	35.60	5.853	5.57	0.50
5	230.0	250.0	82.31	2.185	9.50	1.68
...		...		...		
21	30.0	300.0	12.91	0.026	0.85	0.18

AS CARACTERISTICAS HIDRAULICAS NOS NOS SAO: (nodes data)

node	elevation	node load	head	pressure	min.pressure
NO	COTA TOP. (m)	CONSUMO (l/s)	COTA PIEZ. (m)	ALT.PIEZ. (mca)	A.P.MIN.ADM. (mca)
1	0.00	-141.05	60.50	60.42	20.00
2	0.00	15.54	59.29	59.20	20.00
3	0.00	15.12	41.86	41.80	20.00
...		...		...	
15	0.00	4.53	42.66	42.60	20.00
16	3.80	2.96	54.62	50.75	16.20

O NO. DE RESERVATORIOS E: 1

O ERRO DE FECHO MAXIMO E 0.000 cm NA MALHA 6 (closing error)

PARA FINS COMPARATIVOS O "CUSTO" DAS TUBAGENS E DE 16256 PONTOS (pipe cost)

O COMPRIMENTO TOTAL DAS TUBAGENS E DE 4570 METROS (pipe lenght)

2. Applications of Microcomputers in the Design and Optimisation of Water Supply and Distribution Systems in Developing Countries

A. R. D. Thorley *and* D. J. Wood

ABSTRACT Microcomputers are providing engineers, designers, and managers with considerably improved methods for the design, optimisation, and operation of water supply and distribution systems. This paper discusses issues relevant to the hydraulic design of systems, the overall financial feasibility of systems, and the economic design of systems, and extensions thereto, with the aid of microcomputers. Topics specifically excluded, though worthy in themselves, are the automatic control of systems and data logging of pressures, flows, power consumptions, etc.

The paper concludes with some representative examples of computer applications with particular software for hydraulic design and financial feasibility.

1. INTRODUCTION

The bottom line in most development projects is cost commensurate with meeting the appropriate technical specification. Irrigation and water supply systems are no exception. Schemes need to be evaluated technically and short-listed for the assessment of economic aspects of installation and operation together with possible fault conditions and the need for safety measures.

Prior to the advent of computers engineers designing quite simple systems relied extensively on experience and very crude theoretical models. Even allowing for the gross assumptions made in the theoretical models, they still involved laborious and time-consuming calculations. Computers dispense with much of the routine labour and, perhaps more importantly, enable much more accurate theoretical models to be developed.

These points can be discussed more meaningfully by focussing attention on some relevant and typical applications such as:

- the hydraulic and economic design of new systems
- the appraisal of existing systems and options for uprating

- optimisation of operating strategies
- route planning for tree systems and trunk pipelines

2. HYDRAULIC ANALYSIS AND DESIGN OF PIPE NETWORKS

The basic equations, i.e. laws of conservation of energy and of mass, describing the steady state hydraulic performance of pipelines are non-linear algebraic equations which cannot be solved directly. They can be expressed in two principal fashions, through being written either in terms of the unknown flowrates in the pipes, or in terms of the unknown hydraulic grades at junction nodes throughout the system.

Several algorithms have been proposed for solving the equations and these techniques are in extensive use today (Wood, 1981). One of the most commonly used techniques is the Hardy Cross method (Cross, 1936) which has been widely used for solving both the loop and node network equations. This method involves computing a flow or pressure adjustment which tends to satisfy a single energy or continuity relation for a given loop or node. The iterative adjustment of the unbalanced network equations for each individual loop or junction node, on a sequential basis, means that even microcomputers with limited memory can be used.

More recently, however, simultaneous solutions of the linearised network equations have been devised. These methods have been formulated in terms of the flowrate in each pipe (requiring one equation for each pipe), the hydraulic grades at each junction node (requiring one equation for each node), and the headloss around each loop (requiring one equation for each loop).

A comparative study (Wood, 1981) of the five methods have shown that the simultaneous methods formulated in terms of the flowrate (simultaneous pipe method) or in terms of the headloss around each loop (simultaneous path method) were highly reliable. The other three methods exhibited significant convergence problems and the frequency of problems increased as larger systems were analysed.

Since the simultaneous pipe method requires the simultaneous solution of large equation sets the authors have concluded that the simultaneous path method offers the best procedure for the hydraulic analysis of pipe networks when using microcomputers. They have adopted this technique (Wood and Thorley, 1985) for the analysis of complex networks as it provides highly reliable solutions and yet uses relatively small equation sets.

Although mainframe computers provide the opportunity to improve the theoretical models and automate the calculations, microcomputers have provided the incentive to produce computer codes that are simple to use and despite producing vast quantities of results, enable those of interest to be selected out and displayed in graphical or tabular form (Thorley and Wood, 1985), as will be illustrated later.

2.1. Desirable Features of a Hydraulic Analysis Code

Any hydraulic analysis model should allow consideration of general network configurations with no restrictions on the location of pumps, reservoirs, and similar storage arrangements, as well as accommodating various other hydraulic components. To achieve this each pipe section should be capable of including pumps,

various valves and fittings which cause concentrated energy losses (i.e. 'minor' losses), and connections to constant pressure supplies or discharges. Pump modelling should allow the use of actual pump performance data relating head and efficiency to flowrate.

Data preparation and entry should be simple, with easy editing to provide for additions, changes, and deletions. The user should also be able to select the most appropriate and convenient unit system for his needs and to work in terms of either Hazen-Williams or Darcy Weisbach (i.e. Colebrook-White) friction factors.

Although the computer cannot check the accuracy of pipe lengths and diameters it should undertake a geometric data check of pipe and node numbers to ensure that the pipe system does connect up - and print out a warning if not.

In addition to performing single steady state analyses, the code should provide the option to repeat the analyses with either individual or global changes to the system data (i.e. pipeline and component data, demand changes, etc.). An extension of this feature is the basis for extended period simulations which can model system behaviour over several hours. Reservoir and elevated storage tank levels can vary, pressure regulating and pressure sustaining valves control the behaviour of sections of networks, whilst pressure switches control the operating status of booster pumps as the demand on the network varies. Rapidly changing flows and hydraulic transients are discussed in detail in a companion paper.

The results yielded by a hydraulic analysis, especially for a large network, are quite voluminous and embrace pressure heads, hydraulic grades, flows (and velocities), frictional pressure drops, pumping pressures, efficiency and power requirements, changing reservoir and tank levels together with observations on pipelines and pumps switching on or off, etc. etc. The user needs, therefore, to have control over the nature and volume of the results output.

This can, in fact, be in two stages - firstly, the data stored in a results file on disk can be controlled - and secondly, excerpts can be pulled out from these results files for selective display in graphical or tabular form.

Graphical output should include both geometrical and performance data. In the former case, schematics of the network are produced on the computer screen, with the facility to copy them to a printer. Zoom-in features should enable selected parts of the system to be enlarged. Contours of pipe elevations, hydraulic grades and pressure heads should be capable of superposition over the network display and highlighting should be employed to enable high or low flows, pressure heads and headlosses to be picked out according to user specified threshold values.

The graphical representation of performance data is mainly appropriate for extended period simulations though can include plots of hydraulic grade lines and pipeline profiles. The base line for the time simulations would normally be several hours, but the vertical axis is whatever is desired - pressure heads, system demand, tank levels, etc. Alternatively, tabulation may be used. When dealing with power requirements for the pumps variable electricity costs should be permitted to enable different operating strategies to be investigated.

3. ECONOMIC DESIGN OF NEW SYSTEMS

The total cost of a system comprises its design and installation costs plus the maintenance and operating costs over its design life. The designers' costs would normally be fixed, but there is scope for optimising the capital costs of the installation and its operating costs. This may be illustrated in its simplest form by the compromise required between installing small bore less expensive pipes requiring high pumping costs on the one hand, and larger and more expensive pipes but requiring less pumping power and hence lower operating costs on the other.

The true position is, of course, much more complex than this, especially in developing countries where systems are being installed with an anticipated life of 30-50 years but where, in most cases, the future demand is little more than speculation and a belief, (no doubt, reasonable), that it will increase. To provide flexibility to meet this uncertain future, the trend is to install pipelines that are larger than immediate and near-future demands would normally require, but provide for the expansion of pumping stations in the light of experience and changing circumstances.

This is not to imply that optimum operating conditions are ignored. The escalating cost of energy requires that supply and distribution networks should be operated as cheaply and efficiently as possible. Operating procedures should be devised to achieve the minimum cost per unit volume of water supplied commensurate with maintaining proper safety and reliability standards.

The price of electricity often varies according to the time of day. When additional bulk discounts can be negotiated for off-peak running of pumps, elevated storage tanks and reservoirs to provide for the maximum demands and fire emergencies become very attractive, environmental and space considerations permitting.

4. ENHANCEMENT OF EXISTING SYSTEMS

A common requirement is the design of an extension to an existing system and/or remedial action to overcome operational and performance deficiencies. A computer model of the existing system should, in principle, enable the source of the weaknesses to be identified together with an appropriate solution. However, in contrast to the position some years ago when the main weaknesses in mathematical models were the basic theory and solution techniques employed, the present major area of uncertainty is the quality of the geometrical and physical data for the pipe network.

Specific examples of these uncertainties include the actual lengths of pipelines, effective diameters and pipe roughnesses, valve settings, leakage rates, and customer demands. For a computer model to be relied on to truly simulate the performance of an existing network, like any sophisticated instrument system, it must be calibrated. This requires that synchronised measuremenets of a selection of pressures and flows under known and controlled conditions must be taken, together with reservoir and tank levels.

The pressure measurements should ideally be at various points in the system and not restricted to pumping stations. Similarly, the flowrates should include those in individual lines as well as pumping station throughput, discharges to principal consumers and to/from reservoirs.

Leakages and blockages aside, pipe roughnesses provide the main uncertainty in older networks. Data of the foregoing type are essential for the designer to calibrate his model, supplemented by visual inspection. Damaged and renovated mains provide opportunities for visual inspection and modern video and t.v. camera inspection techniques enable quite small pipelines to be examined.

Should the computer model still yield discrepancies between observed and theoretical results, these anomalies should be investigated, and could be due to:

i) a badly leaking main causing a significant local drop in pressure.

ii) a group of smaller leaks within a defined region of the network - indicated by a higher than expected flow to that region,

and iii) a blockage in a pipeline, or a long forgotten valve partially closed.

Once the model has been calibrated, and the weaknesses defined, the necessary improvements, extensions, and upgrading can be evaluated. These can include:

i) the replacement or parallel laying of pipelines.

ii) the additions of cross-connections between portions of the network.

iii) construction of new reservoirs and storage tanks.

and iv) the uprating of pumping stations and the addition of booster pumping stations.

5. ROUTE PLANNING AND LINE SIZING

Much of the foregoing discussion of computer applications has referred to the analysis of either existing or new systems. The term 'analysis' implies that even the new system already exists in the mind (or on the drawing board) of the engineer, i.e. the design has already been performed 'long-hand' and the computer is being used to check if it is satisfactory. The ideal situation is that the design process should be set in computer code.

Normally, the basic design requirement is that water has to be transferred from a known source, or sources, to specific points and in predetermined quantities. This leads to the need for routes to be defined, and hence pipe lengths, but also for pipe diameters, location and power requirements for pumping stations and the location and capacity of elevated storage tanks to be decided. Some progress is being made in expressing the engineering judgment and logic involved in the decisions required in deciding these issues in a form that is compatible with the way computers work.

When the pipeline routes are predetermined, e.g. by virtue of geographical or other features such as having to follow roads or railways, the task can be relatively straightforward. For example, if, as design criteria, it is possible to specify the required flows and acceptable headloss rate the relevant equations can be solved using either Hazen-Williams or Darcy Weisbach friction factors to provide line diameters, even for looped networks.

If, on the other hand, the starting point is a data set on sources and demands, together with available pipeline sizes and laying costs, procedures have been devised (del Puerto and Hebert, 1985) for defining the least cost design of trunk pipelines and tree structured systems of a modest size. These techniques can also be used for the basic design of a trunk distribution network by adding in the cross-connections as a secondary design phase and then resorting to an analysis to confirm the adequacy or otherwise of the result.

6. EXAMPLE NETWORK ANALYSES

To illustrate the utilisation of microcomputers for analysing typical water distribution systems and for presenting tabular and graphical results some representative output is now presented.

6.1. Hydraulic Analysis

The first example is based on a microcomputer model, using the SIMNET (Wood and Thorley, 1983: Thorley and Wood, 1985) suite of programs, of the trunk distribution system for the city of Djakarta in Indonesia. The system, as modelled, comprises 117 pipes, 87 junction nodes, 6 pumping stations (including 4 booster stations) and 8 supply reservoirs and storage tanks. The pipelines have diameters in the range of 150 - 2000 mm and lengths from 15 - 10,000 m. In its present form the model includes lines that are in process of construction and hence, for the purpose of hydraulic analysis, are shut off.

Table 1 provides an example of the system data describing the network - the main omission, due to lack of space, being the performance data for 6 types of pump. Figure 1 is a copy of the computer generated layout of the network which closely resembles the actual physical layout. The dotted lines are those referred to above as being under construction.

At the time of writing full calibration of the model has not been possible due to lack of reliable information from site of synchronised spot checks on pressures and flowrates. For this reason the pipe roughnesses are all shown (Table 1) as being 0.5 mm. Insofar as some pipelines have been visually inspected this is not unreasonable. Similarly, it is known that a few valves are only partly open, to control pressures in some zones, and are represented by large minor losses - e.g. see pipelines 5 and 112. Ground level in Djakarta is low-lying and fairly flat. It is also known that supply pressures are low.

Table 2 is an excerpt at a time of 4 hours into a 12 hour simulation of this network. It serves to illustrate the type and detail of the results available from a hydraulic analysis. These include flowrates, velocities and headlosses in pipelines, together with energy inputs in the form of pump heads (see lines 82, 83). Data on junction nodes include the local demand (or supply to the system), hydraulic grades and pressure heads. Pump performance data are summarised in a table as also are maximum and minimum values of velocity (not shown), headloss/1000 metres and pressure heads.

In addition to the detailed results tables which can often contain vast quantities of information - so much in fact that critical features may be obscured by the sheer volume - selected results can be extracted and displayed in graphical and tabular form.

Figure 2, for example, illustrates the superposition of contours of pressure head on the geometric layout of the pipe network, enabling the weaker areas to be identified. This figure, being for a large


```
*** SIMNET - PIPE NETWORK ANALYSIS PROGRAM - VERSION 86/1P ***

12 Hour Simulation of the PDAM Jaya System

DATE OF THIS SIMULATION:  15th June 1987
INPUT DATA FILE NAME      = pdex2
OUTPUT DATA FILE NAME     = pdex2out
PIPEVIEW DATA FILE NAME   = pd2rpipe


NUMBER OF PIPES              =  117
NUMBER OF JUNCTION NODES     =  87
   FLOW UNITS                = Litres/second
   PRESSURE UNITS            = m
   RELATIVE DENSITY OF THE FLUID = 1
   KINEMATIC VISCOSITY       =  .0000013

CLOSED LINES -  24   84   85   17   98   81   116

**** SUMMARY OF INPUT DATA ***
```

PIPE NO.	NODE #1	NODE #2	LENGTH (m)	DIAM. (mm.)	ROUGH -NESS	SUM-M FACT.	PUMP TYPE	PGN GRADE
1	66	1	50.0	1000.0	0.500	0.0	0.0	
2	2	3	4600.0	800.0	0.500	0.0	0.0	
3	3	4	3500.0	600.0	0.500	0.0	0.0	
4	4	5	1100.0	600.0	0.500	0.0	0.0	
5	66	2	500.0	1000.0	0.500	25.0	0.0	
6	1	10	450.0	900.0	0.500	0.0	0.0	
55	66	24	500.0	1200.0	0.500	0.0	0.0	
56	24	25	3150.0	1000.0	0.500	0.0	0.0	
57	25	26	1450.0	900.0	0.500	0.0	0.0	
58	27	51	2700.0	600.0	0.500	0.0	0.0	
59	51	44	2400.0	600.0	0.500	0.0	0.0	
60	54	47	1900.0	400.0	0.500	0.0	0.0	
61	44	45	40.0	600.0	0.500	0.0	0.0	
62	0	45	10000.0	600.0	0.500	0.0	0.0	44.0
101	76	0	550.0	350.0	0.500	0.0	0.0	27.0
102	77	76	6000.0	350.0	0.150	0.0	0.0	
103	76	78	1200.0	300.0	0.500	0.0	0.0	
104	9	78	15.0	500.0	0.500	0.0	0.0	
105	78	79	450.0	300.0	0.500	0.0	0.0	
106	79	75	800.0	350.0	0.500	0.0	0.0	
107	79	80	800.0	400.0	0.500	0.0	0.0	
108	80	7	15.0	400.0	0.500	0.0	0.0	
109	22	81	1750.0	400.0	0.500	0.0	0.0	
110	83	80	1150.0	400.0	0.500	0.0	0.0	
111	82	75	2800.0	400.0	0.500	0.0	0.0	
112	1	83	1600.0	300.0	0.500	30.0	0.0	
113	81	83	1650.0	400.0	0.500	0.0	0.0	
114	9	84	1700.0	350.0	0.500	0.0	0.0	
115	85	86	3900.0	1000.0	0.500	0.0	0.0	
116	86	34	2200.0	800.0	0.500	0.0	0.0	
117	87	40	7000.0	800.0	0.500	0.0	0.0	

TABLE 1. Extract from the full computer output of the system data for the 117-pipe model of the PDAM Jaya network.

JUNCT. NO.	DEMAND	ELEVATION
1	0.0	5.5
2	0.0	5.5
3	0.0	4.1
4	0.0	3.5
5	0.0	2.5
6	155.0	6.2
7	0.0	6.5
8	0.0	7.5
9	15.0	7.9
10	0.0	5.5
11	0.0	4.5
12	55.0	4.2
13	185.0	3.8
14	0.0	4.0
15	100.0	4.2
16	55.0	1.2
17	115.0	1.2
	↓	↓
28	25.5	4.6
29	0.0	4.6
30	40.0	2.2
31	75.0	1.8
32	105.0	1.8
33	65.0	4.4
34	0.0	4.2
35	45.0	1.1
36	145.0	0.9
37	0.0	0.9
38	0.0	0.9
39	125.0	1.4
40	75.0	1.5
41	0.0	8.2
42	125.0	8.2
43	0.0	8.4
	↓	↓
80	0.0	6.5
81	0.0	10.0
82	0.0	10.0
83	0.0	10.0
84	60.0	8.5
85	0.0	2.5
86	0.0	2.0
87	0.0	1.5

AN EPS SIMULATION IS SPECIFIED
SIMULATION PERIOD = 12 – TIME INCREMENT = 1

--- TANK DATA ---

PIPE NO.	MAX. EL.	MIN. EL.	DIAMETER	INIT. EL.	EXT. Q(IN)
62	46.0	40.0	42.7	44.0	0.0
63	46.0	40.0	42.7	44.0	0.0

THERE ARE CHECK VALVES IN THE FOLLOWING LINES: 46 103 104 109

TABLE 1 (Continued)

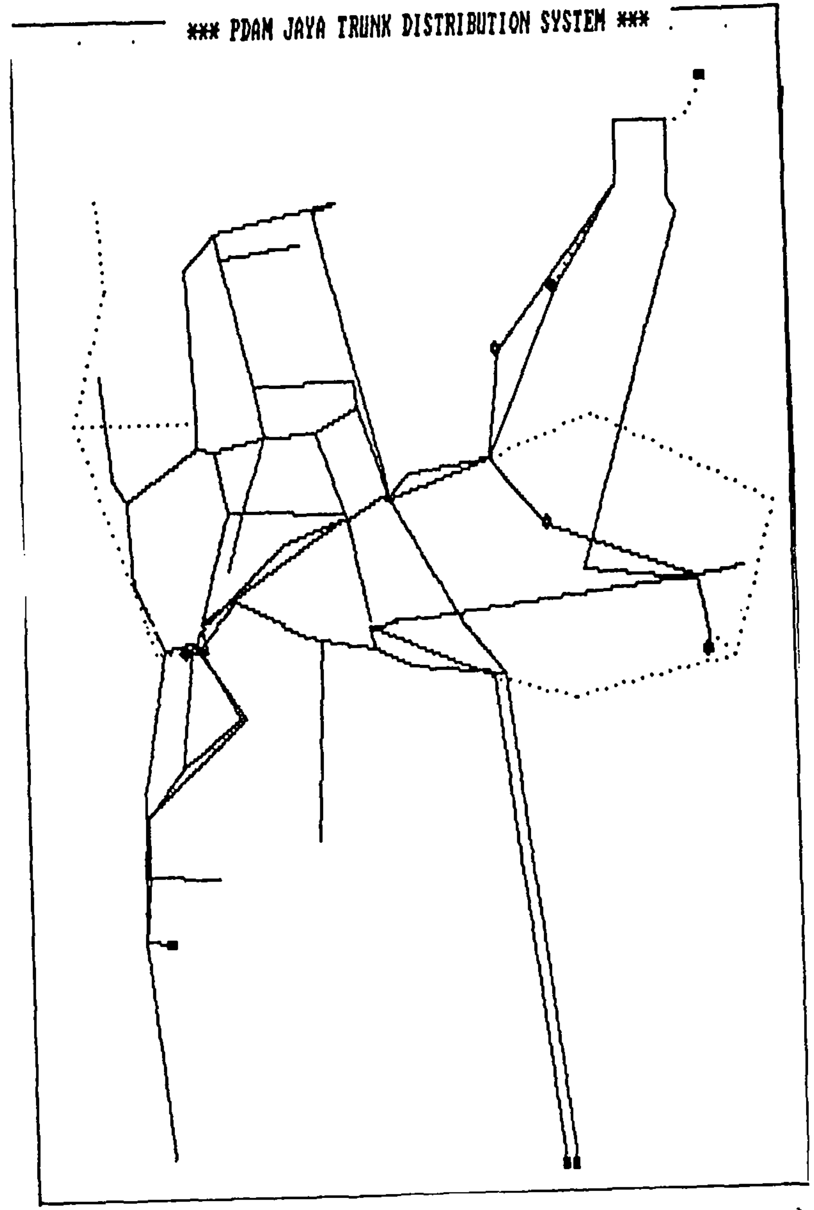

FIGURE 1. Computer generated layout of the PDAM Jaya network. Lines under construction are shown dotted.

EPS SIMULATION - TIME = 4 HOURS
**** THE RESULTS FOR THIS SIMULATION FOLLOW ****

NO. OF TRIALS = 1 - ACCURACY ATTAINED = .0045

PIPE NO.	NODE #1	NODE #2	FLOW RATE	HEAD LOSS	MINOR LOSS	PUMP HEAD	LINE VELOCITY	HL 1000
1	66	1	132.54	0.00	0.00	0.00	0.17	0.03
2	2	3	0.00	0.00	0.00	0.00	0.00	0.00
3	3	4	0.00	0.00	0.00	0.00	0.00	0.00
4	4	5	0.00	0.00	0.00	0.00	0.00	0.00
5	66	2	0.00	0.00	0.00	0.00	0.00	0.00
6	1	10	201.51	0.05	0.00	0.00	0.32	0.11
7	10	6	36.96	0.02	0.00	0.00	0.07	0.01
8	6	7	-79.29	0.01	0.00	0.00	0.16	0.03
9	7	8	56.25	0.06	0.00	0.00	0.20	0.07
			↓				↓	
78	63	41	191.37	0.04	0.00	0.00	0.97	1.99
79	41	42	93.75	0.37	0.12	0.00	0.48	0.50
80	67	63	293.27	0.51	0.17	0.00	0.58	0.41
LINE NO. 81 IS SHUT OFF								
82	0	66	70.61	0.00	0.00	11.63	0.02	0.00
83	0	67	293.27	0.00	0.00	14.78	0.09	0.00
LINE NO. 84 IS SHUT OFF								
LINE NO. 85 IS SHUT OFF								
86	17	74	22.97	0.21	0.00	0.00	0.18	0.10
			↓				↓	
108	80	7	135.54	0.05	0.00	0.00	1.08	3.21
109	22	81	10.25	0.04	0.00	0.00	0.08	0.02
110	83	80	8.51	0.02	0.00	0.00	0.07	0.02
111	82	75	-67.23	2.28	0.00	0.00	0.53	0.92
112	1	83	-1.74	0.01	0.00	0.00	0.02	0.00
113	81	83	10.25	0.04	0.00	0.00	0.08	0.02
114	9	84	45.00	1.26	0.00	0.00	0.47	0.74
115	85	86	0.00	0.00	0.00	0.00	0.00	0.00
LINE NO. 116 IS SHUT OFF								
117	87	40	101.90	0.37	0.00	0.00	0.20	0.05

SUMMARY OF PUMP OPERATION

PIPE NO.	PUMP TYPE	PUMP FLOW	PUMP HEAD	USEFUL POWER	EFFIC-IENCY	TOTAL KWH
26	1	430.31	11.70	49.38	0.80	266.26
71	4	127.70	6.66	8.33	0.80	43.38
82	2	70.61	11.63	8.06	0.80	68.55
83	3	293.27	14.78	42.52	0.80	226.46
93	5	32.58	7.84	2.50	0.80	12.43
95	6	62.90	5.10	3.14	0.80	16.16

THE TOTAL POWER USED TO THIS TIME = 633.25 KWH

TABLE 2. Brief extract from a hydraulic analysis of the Djakarta distribution system.

JUNCTION NO.	ELEVATION (m)	DEMAND	PRESSURE HEAD (m)	HYDRAULIC GRADE
1	5.5	0.0	7.6	13.1
2	5.5	0.0	7.6	13.1
3	4.1	0.0	9.0	13.1
4	3.5	0.0	9.6	13.1
5	2.5	0.0	10.6	13.1
6	6.2	116.3	6.9	13.1
7	6.5	0.0	6.6	13.1
8	7.5	0.0	5.5	13.0
9	7.9	11.2	5.0	12.9
↓			↓	
44	8.9	0.0	13.6	22.5
45	18.0	0.0	4.6	22.6
46	18.0	0.0	2.1	20.1
47	8.9	0.0	11.2	20.1
48	9.5	0.0	9.7	19.2
49	4.6	3.8	11.9	16.5
↓			↓	
83	10.0	0.0	3.1	13.1
84	8.5	45.0	3.1	11.6
85	2.5	0.0	13.8	16.3
86	2.0	0.0	14.3	16.3
87	1.5	0.0	13.9	15.4

SUMMARY OF MINIMUM AND MAXIMUM HL/1000

MINIMUMS		MAXIMUMS	
82	0.00	42	34.96
55	0.00	105	5.77
54	0.00	103	5.77
56	0.00	102	4.02
57	0.00	101	3.83

SUMMARY OF MINIMUM AND MAXIMUM PRESSURE HEADS

MINIMUMS		MAXIMUMS	
56	1.92	77	19.03
46	2.15	76	16.69
84	3.11	35	15.29
83	3.12	36	14.86
81	3.16	38	14.81

--- TANK STATUS REPORT ---

PIPE NO.	PIPE Q	EXT. Q	ELEVATION	PROJ. EL.
62	-296.8	0.0	40.9	40.2
63	-314.5	0.0	40.7	40.0

TABLE 2 (Continued)

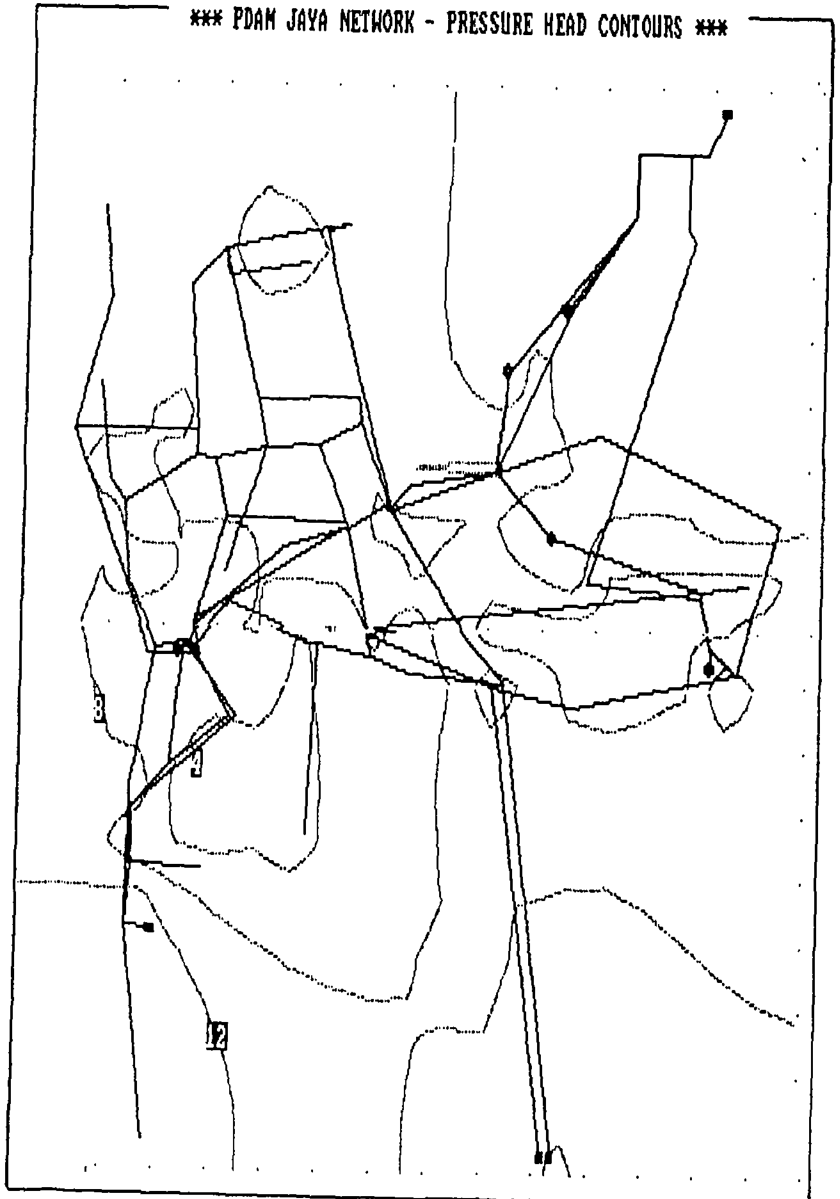

FIGURE 2. Contours of pressure head superimposed on the system layout.

network, really just provides an overview, but by zooming into a part of the network, greater detail is obtained - see Figure 3. Yet closer detail can be obtained, as shown by Figure 4 (for another system) should this be required.

For time simulations it is often preferred to have pertinent data plotted against time and typical examples are shown in Figures 5 and 6. Alternatively, tabular representation can be used as shown in Tables 3 and 4. The former refers to conditions and junction nodes within a network; the latter refers to flows out of and in to variable level reservoirs.

This information is very relevant to the development of strategies for optimising system performance. So too are the economics of pump performance, especially when variable cost power sources are available. Table 5 provides an example of data output available in this respect from modern microcomputer analyses.

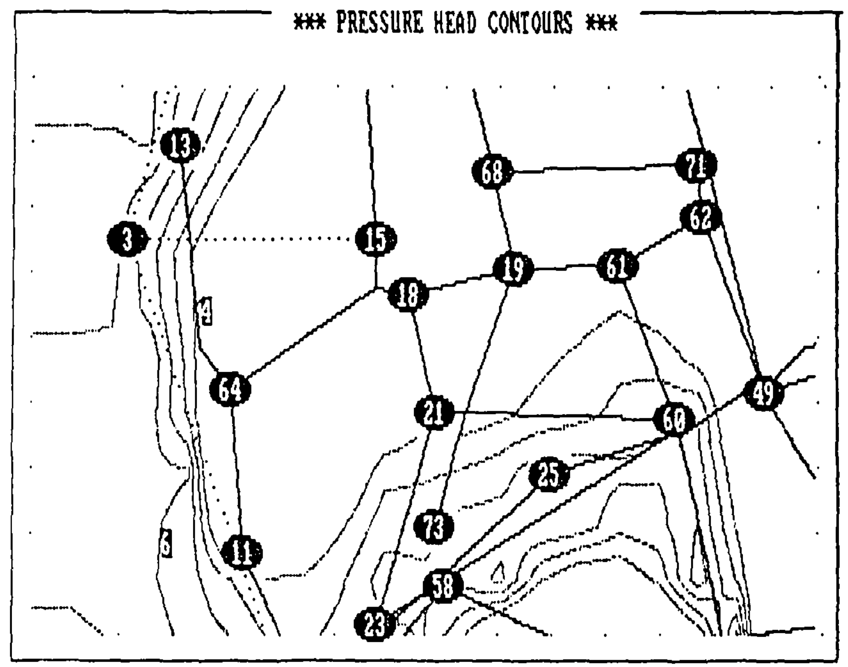

FIGURE 3. Close-up of part of the PDAM Jaya network - see left-centre of Figure 2.

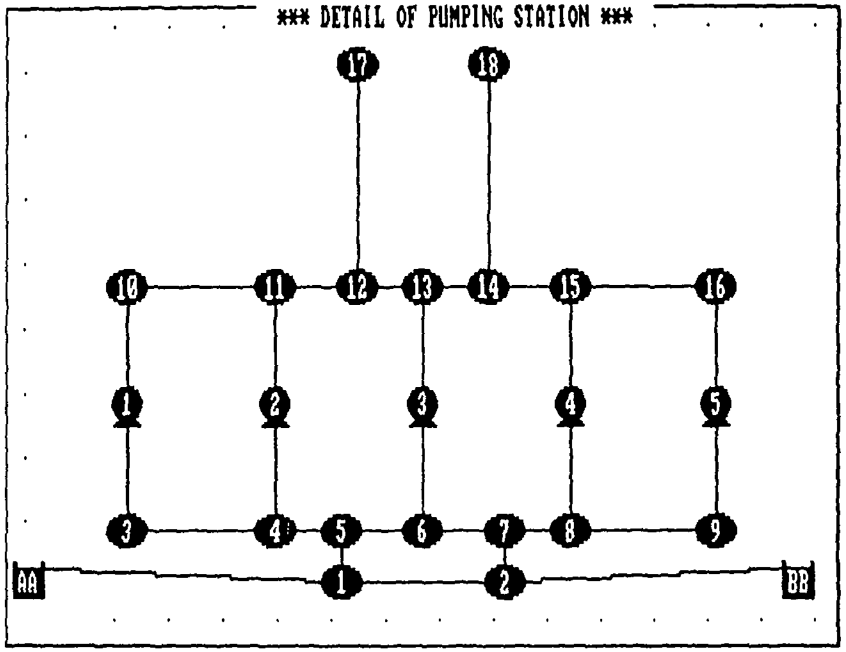

FIGURE 4. Close-up view of the geometric layout of a pumping station.

6.2. Minimum Basic Tree Structures

Another feature of interest for system optimisation is a capability to identify the minimum tree system to provide necessary supplies. This provides the basic skeletal structure of a system to minimise the length of the larger diameter trunk mains required to carry the majority of the flow. Additional secondary links are then added to provide for reliability and for service to customers not on the primary system. Figure 7 is a sample result of such an analysis[7]. This technique is based on operational research techniques so does not necessarily, or directly, provide an optimal solution from economic or hydraulic viewpoints. Nevertheless, it is a stepping stone to more sophisticated techniques which are also available for microcomputers.

These (see del Puerto and Hebert, 1985 for example) make use of data banks containing information on the purchasing and laying costs of commercially available pipes and user-defined design criteria, such as max-min headloss rates, to evaluate the cheapest pipe system to meet required specification. Table 6 (from del Puerto and Hebert, 1985) is a sample result. This particular program is restricted to water systems due to the use of the Hazen-Williams representation of frictional headloss and only tree-type structures may be designed.

***************** TABLE OF HYDRAULIC GRADE LINES AND PRESSURES *************

LOCATIONS FOR JUNCTION NODES

	WOOD ST.		PARK		RIVER		HIGH	
TIME	HGL	PRESSURE	HGL	PRESSURE	HGL	PRESSURE	HGL	PRESSURE
0.00	86.00	563.85	82.74	527.02	83.78	492.12	79.30	380.53
1.00	86.00	563.85	82.74	527.02	83.78	492.12	79.30	380.53
2.00	77.67	482.23	71.35	415.30	70.09	357.88	64.09	239.24
3.00	76.86	474.26	70.57	407.68	69.31	350.26	64.09	231.33
4.00	76.07	466.50	69.81	400.21	68.56	342.83	63.30	223.62
5.00	69.36	400.69	57.62	280.65	60.45	263.33	52.17	114.42
5.00	71.02	417.04	60.12	305.20	62.66	285.03	55.80	150.05
6.00	69.91	406.11	59.07	294.91	61.66	275.17	54.66	138.85
7.00	68.82	395.41	58.04	284.81	60.67	265.51	53.54	127.93
7.85	62.91	337.44	46.46	171.27	50.41	164.89	40.41	-0.87
8.00	62.71	335.53	46.30	169.67	50.29	163.72	40.22	-2.79
8.81	39.67	109.54	31.27	22.21	35.01	13.82	27.18	-130.60
8.99	20.25	-80.87	8.58	-200.30	17.23	-160.58	3.87	-359.27
9.00	53.17	241.91	47.03	176.85	47.64	137.72	42.81	22.62
10.00	75.60	461.95	73.91	440.42	75.58	411.66	72.26	311.50
10.00	74.41	450.21	72.71	428.65	74.71	403.18	69.96	288.89
11.00	75.10	456.97	73.34	434.80	74.95	405.56	70.14	290.70
12.00	75.40	459.90	73.31	434.57	75.30	408.99	70.47	293.96

TABLE 3. Hydraulic grades and pressure heads at four locations.

****************** TABLE OF FGN INFLOWS (OUTFLOWS) AND GRADES *******

LOCATIONS FOR FIXED GRADE NODES

	SOURCE B		SOURCE D		SOURCE E	
TIME	HGL	FLOW	HGL	FLOW	HGL	FLOW
0.00	81.00	0.00	81.00	0.00	81.00	0.00
1.00	81.00	0.00	81.00	0.00	81.00	0.00
2.00	81.00	71.93	81.00	64.41	81.00	98.10
3.00	79.98	69.50	80.09	63.92	79.61	94.53
4.00	79.00	67.17	79.18	63.39	78.27	91.09
5.00	78.05	120.78	78.29	114.48	76.99	145.09
5.00	78.05	107.67	78.29	107.22	76.99	127.69
6.00	76.53	104.24	76.77	106.28	75.18	123.89
7.00	75.05	100.93	75.27	105.31	73.43	120.22
7.85	73.84	136.70	74.00	121.98	72.00	0.00
8.00	73.55	136.04	73.74	121.88	72.00	0.00
8.81	72.00	0.00	72.34	142.33	72.00	0.00
8.99	72.00	0.00	72.00	0.00	72.00	0.00
9.00	72.00	0.00	72.00	0.00	72.00	0.00
10.00	72.00	-75.09	72.00	-24.90	72.00	-35.70
10.00	72.00	-60.39	72.00	-9.87	72.00	-17.67
11.00	72.85	-58.11	72.14	-10.56	72.25	-17.58
12.00	73.68	-50.34	72.29	-13.20	72.50	-18.72

TABLE 4. Hydraulic grades and flowrates at three variable level sources.

************* PUMP OPERATION RESULTS *************

TIME = 0 HRS - POWER COSTS = 3.7 pence/kWh

PUMP LABEL	FLOW RATE	PUMP HEAD	USEFUL POWER	EFFIC-IENCY	REQ. KWH	COST PER HR	TOTAL COSTS
P1	346.63	70.58	239.92	0.71	336.96	12.47	0.00
P2	332.37	68.95	224.73	0.68	328.21	12.14	0.00
P3	0.00	0.00	0.00	0.10	0.00	0.00	0.00

TOTAL COST PER HOUR (pounds) = 24.61
TOTAL COSTS TO THIS POINT (pounds) = 0

TIME = 1 HRS - POWER COSTS = 3.7 pence/kWh

PUMP LABEL	FLOW RATE	PUMP HEAD	USEFUL POWER	EFFIC-IENCY	REQ. KWH	COST PER HR	TOTAL COSTS
P1	346.63	70.58	239.92	0.71	336.96	12.47	12.47
P2	332.37	68.95	224.73	0.68	328.21	12.14	12.14
P3	0.00	0.00	0.00	0.10	0.00	0.00	0.00

TOTAL COST PER HOUR (pounds) = 24.61
TOTAL COSTS TO THIS POINT (pounds) = 24.61

↓ ↓

TIME = 5 HRS - POWER COSTS = 4.5 pence/kWh

PUMP LABEL	FLOW RATE	PUMP HEAD	USEFUL POWER	EFFIC-IENCY	REQ. KWH	COST PER HR	TOTAL COSTS
P1	427.39	59.43	249.07	0.55	453.52	20.41	84.63
P2	0.00	0.00	0.00	0.10	0.00	0.00	90.21
P3	488.53	67.99	325.75	0.66	491.07	22.10	0.00

TOTAL COST PER HOUR (pounds) = 42.5
TOTAL COSTS TO THIS POINT (pounds) = 174.84

TIME = 6 HRS - POWER COSTS = 4.5 pence/kWh

PUMP LABEL	FLOW RATE	PUMP HEAD	USEFUL POWER	EFFIC-IENCY	REQ. KWH	COST PER HR	TOTAL COSTS
P1	431.65	58.79	248.87	0.54	462.48	20.81	105.44
P2	0.00	0.00	0.00	0.10	0.00	0.00	90.21
P3	492.45	67.57	326.34	0.66	493.08	22.19	22.19

TOTAL COST PER HOUR (pounds) = 42.99
TOTAL COSTS TO THIS POINT (pounds) = 217.84

TABLE 5. Power consumption and operating costs for three pumps during a 12 hour simulation.

↓ ↓

TIME = 7 HRS - POWER COSTS = 4.5 pence/kWh

PUMP LABEL	FLOW RATE	PUMP HEAD	USEFUL POWER	EFFIC-IENCY	REQ. KWH	COST PER HR	TOTAL COSTS
P1	435.80	58.17	248.59	0.53	471.62	21.22	126.67
P2	0.00	0.00	0.00	0.10	0.00	0.00	90.21
P3	496.25	67.16	326.86	0.66	495.01	22.28	44.46

TOTAL COST PER HOUR (pounds) = 43.49
TOTAL COSTS TO THIS POINT (pounds) = 261.34

↓ ↓

TIME = 8 HRS - POWER COSTS = 4.5 pence/kWh

PUMP LABEL	FLOW RATE	PUMP HEAD	USEFUL POWER	EFFIC-IENCY	REQ. KWH	COST PER HR	TOTAL COSTS
P1	467.89	53.24	244.28	0.43	563.03	25.34	151.93
P2	0.00	0.00	0.00	0.10	0.00	0.00	90.21
P3	532.69	63.11	329.68	0.64	513.06	23.09	67.55

TOTAL COST PER HOUR (pounds) = 48.42
TOTAL COSTS TO THIS POINT (pounds) = 309.69

↓ ↓

TIME = 11 HRS - POWER COSTS = 3.7 pence/kWh

PUMP LABEL	FLOW RATE	PUMP HEAD	USEFUL POWER	EFFIC-IENCY	REQ. KWH	COST PER HR	TOTAL COSTS
P1	391.86	64.54	248.04	0.63	392.55	14.52	236.29
P2	373.39	64.78	237.21	0.61	387.40	14.33	104.55
P3	0.00	0.00	0.00	0.10	0.00	0.00	108.76

TOTAL COST PER HOUR (pounds) = 28.85
TOTAL COSTS TO THIS POINT (pounds) = 449.59

TIME = 12 HRS - POWER COSTS = 3.7 pence/kWh

PUMP LABEL	FLOW RATE	PUMP HEAD	USEFUL POWER	EFFIC-IENCY	REQ. KWH	COST PER HR	TOTAL COSTS
P1	389.48	64.88	247.79	0.64	389.14	14.40	250.69
P2	371.78	64.95	236.81	0.62	384.68	14.23	118.78
P3	0.00	0.00	0.00	0.10	0.00	0.00	108.76

TOTAL COST PER HOUR (pounds) = 28.63
TOTAL COSTS TO THIS POINT (pounds) = 478.22

TABLE 5. (Continued)

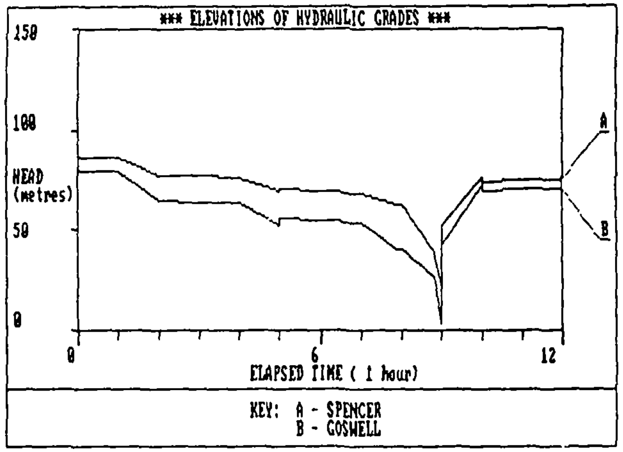

FIGURE 5. An example time plot showing variation of the hydraulic grade lines on two nodes.

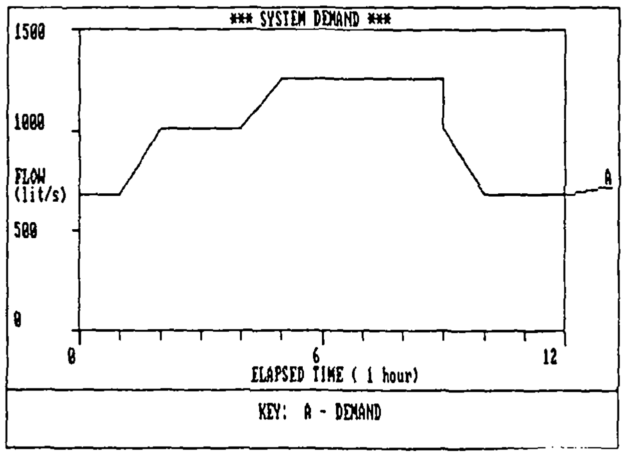

FIGURE 6. Fluctuation in the demand on a water supply system.


```
T I T L E      :  BRANCH NETWORK SAMPLE DESIGN
NO. OF LINKS   :  13
NO. OF NODES   :  14
PEAK FACTOR    :  1
MIN  HL/KM     :  .05
MAX  HL/KM     :  20
RESIDUAL HEAD  :  10
```

PIPE NO.	NODE FROM	NODE TO	LENGTH	DIA	HWC
1	1	2	500.00	0	0
2000	2	3	500.00	100	110
3	2	4	500.00	0	0
4	2	6	500.00	100	110
5	6	5	500.00	0	0
6	6	7	500.00	0	0
7	6	9	500.00	0	0
8	9	8	500.00	0	0
9	10	9	500.00	0	0
10	9	12	500.00	0	0
11	12	11	500.00	0	0
12	12	13	500.00	0	0
13	14	10	500.00	0	0

NODE #	FIX	FLOW	ELEVATION
1	0.0	0.000	25.00
2	0.0	-3.000	0.00
3	0.0	-3.000	0.00
4	0.0	-3.000	0.00
5	0.0	-3.000	0.00
6	0.0	-3.000	0.00
7	0.0	-3.000	0.00
8	0.0	-3.000	0.00
9	0.0	-3.000	0.00
10	0.0	-3.000	0.00
11	0.0	-3.000	0.00
12	0.0	-3.000	0.00
13	0.0	-3.000	0.00
14	1.0	10.000	0.00

REFERENCE NODE	GRADE LINE
1	25.00

TABLE 6. Specimen output for 'least cost' layout of a branched pipeline for irrigation or water supply (from Ref.5).

AVAILABLE PIPES :

DIAM (MM)	HWC	UNIT COST
75	100	75.00
100	110	120.00
150	110	200.00
200	110	300.00
250	120	430.00

PIPE no.	NODES from	to	FLOW (lps)	DIAM (mm)	HWC	HLOSS (m)	HL/KM (m)	LENGTH (m)	COST
1	1	2	26.000	200	110	2.64	5.28	500.00	150,000.00
2000	2	3	3.000	100	110	1.42	2.84	500.00	
3	2	4	3.000	75	100	6.87	13.75	500.00	37,500.00
4	2	6	17.000	100	110	2.86	5.73	500.00	
				150	110	2.86	5.73	500.00	100,000.00
5	6	5	3.000	75	100	6.87	13.75	500.00	37,500.00
6	6	7	3.000	75	100	6.87	13.75	500.00	37,500.00
7	6	9	8.000	150	110	1.21	2.42	500.00	100,000.00
8	9	8	3.000	75	100	6.87	13.75	500.00	37,500.00
9	10	9	7.000	100	110	6.81	13.61	500.00	60,000.00
10	9	12	9.000	150	110	1.50	3.01	500.00	100,000.00
11	12	11	3.000	75	100	6.76	13.75	491.82	36,886.42
				100	110	0.02	2.84	8.18	981.73
12	12	13	3.000	75	100	6.76	13.75	491.82	36,886.42
				100	110	0.02	2.84	8.18	981.73
13	14	10	10.000	150	110	1.83	3.66	500.00	100,000.00
								TOTAL =	835,736.38

NODE NO.	FLOW (LPS)	ELEV (M)	H G L (M)	PRESSURE (M)
1S	26.000	25.0	25.0	0.0
2	-3.000	0.0	22.4	22.4
3	-3.000	0.0	20.9	20.9
4	-3.000	0.0	15.5	15.5
5	-3.000	0.0	12.6	12.6
6	-3.000	0.0	19.5	19.5
7	-3.000	0.0	12.6	12.6
8	-3.000	0.0	11.4	11.4
9	-3.000	0.0	13.3	18.3
10	-3.000	0.0	25.1	25.1
11	-3.000	0.0	10.0	10.0
12	-3.000	0.0	16.8	16.8
13	-3.000	0.0	10.0	10.0
14	10.000	0.0	26.9	26.9

S U M M A R Y

DIAM (MM)	LENGTH (M)	COST
75	2,983.6	223,772.83
100	516.4	61,963.45
150	2,000.0	400,000.00
200	500.0	150,000.00
TOTAL =		835,736.25

TABLE 6. (Continued)

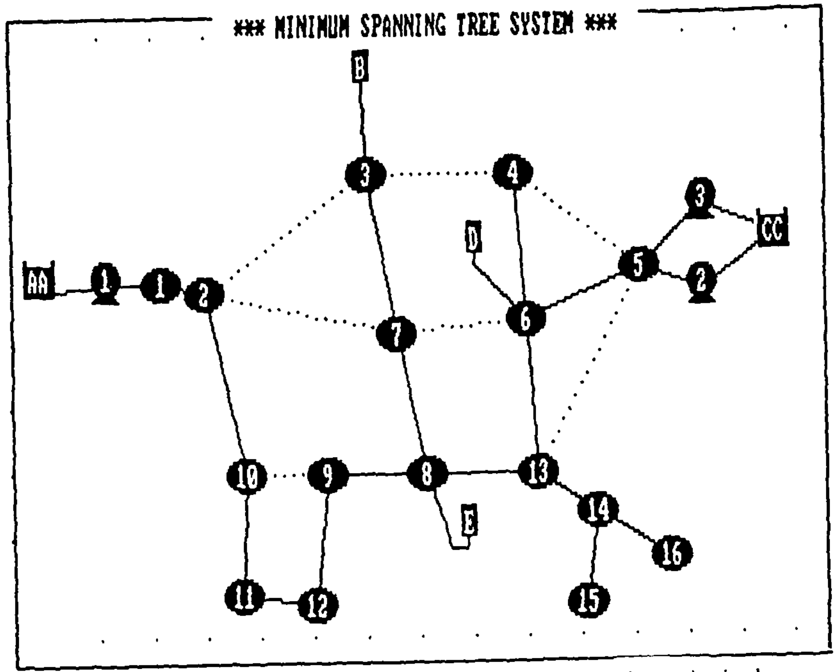

FIGURE 7. Looped water supply system showing the principal minimum length tree-type skeleton in solid lines. Dotted lines indicate secondary connections for security of supply.

6.3. Overall Project Feasibility

As a general rule, the preparation of a complete financial feasibility analysis for a given water supply project is, in itself, an expensive undertaking. Sometimes detailed and costly studies have been undertaken only to discover that the proposed scheme was not so financially attractive after all. The use of slightly less sophisticated models, capable of being run on microcomputers, can help contain the expense of these preliminary studies and yet still provide much useful data.

Results from one such model (Roncesvalles, 1985) using an IBM-PC are shown in Table 7. The scheme used is quite arbitrary but serves to illustrate what can be achieved.

To use this type of model basic assumptions that are made include:

i) a preliminary project scheme has been formulated.
ii) construction costs and other project cost items relevant to the system have been estimated.

Project : *** MUNICIPAL WATER SUPPLY SCHEME ***

SUMMARY OF RESULTS OF FINANCIAL FEASIBILITY ANALYSIS
Project : *** MUNICIPAL WATER SUPPLY SCHEME ***
[Costs in : US dollars]

TOTAL PROJECT COST [TOTAL PROJECT LOAN] >>>------------>			8551118
Project Cost/capita served for 1998 , Design Year			17
Project Cost/capita served for 1991 , First Yr. of Operation			34

For the First Three Years of Operation >>>	1991	1992	1993
TOTAL POPULATION SERVED	250000	300000	400000
Total Water Production, cumd	57072	155252	373395
Total Water Accounted for, cumd	48511	131964	317386
Total Dom/Res Consumption, cumd	29401	77625	171560
TOTAL ANNUAL SYSTEM COST	1545440	2474719	4726178
Annual Sys. Cost/capita served	6	8	12
Annual Sys. Cost/cu.m. produced	.08	.04	.04
Annual Sys. Cost/cu.m. accounted for	9.000001E-02	.05	.04
TOTAL ANNUAL OPERATION & MAINTENANCE COST	425600	1261951	3308264
Annual O & M Cost/capita served	2	4	8
Annual O & M Cost/cu.m. produced	.02	.02	.02
Annual O & M Cost/cu.m. accounted for	.02	.03	.03
% of COSTS to be borne by REVENUES realized from DOM./RES. CONSUMPTION	15	20	25
Equivalent Annual System Cost	231816	494944	1181545
Effective Cost/cu.m.-Dom/Res Cons.	.02	.02	.02
Effective Cost/Capita served	1	2	3
Equivalent Annual O & M Cost	63840	252390	827066
Effective Cost/cu.m.-Dom/Res Cons.	.01	.01	.01
Effective Cost/Capita served	0	1	2

B A S I C I N F O R M A T I O N
Project : *** MUNICIPAL WATER SUPPLY SCHEME ***

A. BACKGROUND INFORMATION

Location of Project:	
Country	Undisclosed
Province/State	Undisclosed
City/Town	Undisclosed
Unit of Currency	US dollars
Ave. No. of Persons/Household	5
Population Growth Rate: % Increase/year	2

B. PROJECT TIMETABLE

Year when Project Planning begins	1987
Year when Construction begins	1989
Year when Operation starts [First year of Operation]	1991
Year when Further expansion of the system will be required [Design Year]	1998
* Project Planning Period, years	2
* Construction Period, years	2
* Design Period, years	7

C. LOAN TERMS

Length of Loan Repayment, years	30
Annual Interest Rate, %	9

Table 7. *Sample Output from a microcomputer feasibility study of a proposed water supply scheme.*

POPULATION SERVED AND WATER DEMAND PROJECTIONS
Project : *** MUNICIPAL WATER SUPPLY SCHEME ***

Year >>	1991 [1]	1992	1993	1998 [2]
TOTAL POPULATION SERVED	250000	300000	400000	500000
A. DOMESTIC / RESIDENTIAL DEMAND:				
%of Tot.Pop'n served w/ House Conn's.	75	85	95	95
Equivalent Population	187500	255000	380000	475000
Unit use, Lpcd	135	300	450	450
Demand Subtotal, cumd	25313	76500	171000	213750
%of Tot.Pop'n served w/ Yard Taps	20	10	2	2
Equivalent Population	50000	30000	8000	10000
Unit use, Lpcd	75	25	25	25
Demand Subtotal, cumd	3750	750	200	250
%of Tot.Pop'n served w/ Pub. Faucets	3	5	3	2
Equivalent Population	7500	15000	12000	10000
Unit use, Lpcd	25	25	30	35
Demand Subtotal, cumd	188	375	360	350
%of Tot.Pop'n served w/ Other types of service	2	0	0	1
Equivalent Population	5000	0	0	5000
Unit use, Lpcd	30	0	0	55
Demand Subtotal, cumd	150	0	0	275
TOTAL DOMESTIC/RESIDENTIAL DEMAND, cumd	29401	77625	171560	214625
B. COMMERCIAL / INDUSTRIAL DEMAND:				
Commercial Demand				
% of Total Dom/Res Demand	25	30	40	35
Equivalent Lpcd	29	78	172	150
Demand Subtotal, cumd	7350	23288	68624	75119
Industrial Demand				
% of Total Dom/Res Demand	30	30	40	40
Equivalent Lpcd	35	78	172	172
Demand Subtotal, cumd	8820	23288	68624	85850
TOTAL COMMERCIAL/INDUSTRIAL DEMAND, cumd	16170	46576	137248	160969
C. PUBLIC DEMAND:				
% of Total Dom/Res Demand	10	10	5	15
Equivalent Lpcd	12	26	21	64
TOTAL PUBLIC DEMAND, cumd	2940	7763	8578	32194
TOTAL ACCOUNTED FOR WATER, cumd	48511	131964	317386	407788
D. UNACCOUNTED FOR WATER:				
% of Total Water Produced	15	15	15	10
Equivalent Lpcd	34	78	140	91
TOTAL UNACCOUNTED FOR WATER, cumd	8561	23288	56009	45310
TOTAL WATER DEMAND [Total Prod'n], cumd	57072	155252	373395	453098

[1] 1st yr. of Op'n; [2] Design yr.

TABLE 7. (Continued)

TABLE
ESTIMATED COST OF CONSTRUCTION
Project : *** MUNICIPAL WATER SUPPLY SCHEME ***

	US dollars
A. COST ESTIMATES FOR NEW FACILITIES	
Source Facilities:	
Wells	450000
Pump Stations	800000
Spring Development	100000
Surface Water Intake	250000
Infiltration Galleries	150000
Reservoirs	1500000
Others	250000
Sub-Total	3500000
Transmission Facilities:	
Raw Water Transmission	150000
Raw Water Pump Stations	35000
Treated Water Transmission	25000
Treated Water Pump Stn's.	40000
Others	15000
Sub-Total	265000
Treatment Facilities:	
Disinfection Equipment	15000
Rapid Sand Filtration	20000
Slow Sand Filtration	15000
Coagulation/Flocculation	10000
Sedimentation	5000
Others	5000
Sub-Total	70000
Distribution Facilities:	
Pipe Network	75000
Booster Pump Stations	20000
Service Connections	18000
Public Faucets	5000
Ground Storage Tanks	7500
Elevated Storage Tanks	8000
Private Storage Tanks	6500
Other Storage Tanks	4000
Others	6000
Sub-Total	150000
Other Facilities:	250000
TOTAL COST OF NEW FACILITIES	4235000
B. CONTINGENCIES (7.5 % of Total Cost of New Facilities)	317625
TOTAL COST OF CONSTRUCTION	4552625

TABLE 7. (Continued)

TOTAL PROJECT COST [TOTAL PROJECT LOAN]
Project : *** MUNICIPAL WATER SUPPLY SCHEME ***

	US dollars
TOTAL COST OF CONSTRUCTION	4552625
Engineering Cost (35 % of Total Const. Cost)	1593419
Legal Cost (7 % of Total Const. Cost)	318684
Land Cost (25 % of Total Const. Cost)	1138156
Other Miscellaneous Costs (5 % of Total Const. Cost)	227631
Sub-Total	7830515
Capitalized Interest	720603
TOTAL PROJECT COST [TOTAL PROJECT LOAN]	8551118

ESTIMATED ANNUAL OPERATION AND MAINTENANCE COSTS
First Three Years of Operation
Project : *** MUNICIPAL WATER SUPPLY SCHEME ***
[Costs in : US dollars]

Year >>>	1991	1992 [1]	1993 [1]
Power and Utilities	120000	355813	932781
Chemicals	60000	177907	466391
Maintenance and Repairs	25000	74128	194329
Salaries and Wages	130000	385464	1010513
Transportation	30000	88953	233195
Other O & M Costs	15000	44477	116598
Sub - Total	380000	1126742	2953807
O & M Contingencies (12 % of Sub-Total)	45600	135209	354457
TOTAL ANNUAL OPERATION AND MAINTENANCE COSTS	425600	1261951	3308264

[1] Cost Escalation Rates: 1991 - 1992 = 9 %; 1992 - 1993 = 9 %

ESTIMATED TOTAL ANNUAL SYSTEM COSTS
First Three Years of Operation
Project : *** MUNICIPAL WATER SUPPLY SCHEME ***
[Costs in : US dollars]

Year >>>	1991	1992	1993
A. DEBT SERVICE			
Amortization due: a) This project	845296	845296	845296
b) Other Previous Loans	120000	120000	100000
TOTAL DEBT SERVICE	965296	965296	945296
B. TOTAL ANNUAL OPERATION & MAINTENANCE COSTS	425600	1261951	3308264
Sub-total	1390896	2227247	4253560
C. CONTINGENCIES (10 % of Total Ann. Sys. Cost)	154544	247472	472618
TOTAL ANNUAL SYSTEM COSTS	1545440	2474719	4726178

TABLE 7. (Continued)

iii) operation and maintenance costs of the system during its first year of operation have been estimated.
iv) the project is to be financed solely from a loan for the full cost.
v) the first three years of operation is the most critical for financial survival, so only these years will be considered.
vi) measures of financial feasibility are based on the costs associated with designing, constructing, and operating the system.

Water demand projections include the domestic, commercial and industrial demands and also include provision for unaccounted water such as leakage losses. The total project cost is the sum of the costs of construction, including items such as land, engineering, legal and other miscellaneous costs, and capitalised interest.

The first page of Table 7 gives an overall view of the project, population served, summary of costs and timetable of the main events. The second part provides greater details of the water demands under different categories of use. The construction costs are estimated next, followed by the operation and maintenance costs for each of the first three years of operation.

7. CONCLUDING REMARKS

It is clear that low cost microcomputers provide an effective tool for engineers to undertake the accurate, efficient, and cost effective design and optimisation, including the financial feasibility, of pipe networks. Information requried by engineers, managers, and designers can be generated and presented in an easily assimilated form, and the reliability of this data is now dependent more on the uncertainty of the basic design information rather than the theoretical and numerical models used for its manipulation.

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RESEARCH STUDIES PRESS LTD.
Letchworth, Hertfordshire, England

JOHN WILEY & SONS INC.
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RESEARCH STUDIES PRESS LTD.
58B Station Road, Letchworth, Herts. SG6 3BE, England

Marketing and Distribution:

Australia, New Zealand, South-east Asia:
Jacaranda-Wiley Ltd., Jacaranda Press
JOHN WILEY & SONS INC.
GPO Box 859, Brisbane, Queensland 4001, Australia

Canada:
JOHN WILEY & SONS CANADA LIMITED
22 Worcester Road, Rexdale, Ontario, Canada

Europe, Africa:
JOHN WILEY & SONS LIMITED
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North and South America and the rest of the world:
JOHN WILEY & SONS INC.
605 Third Avenue, New York, NY 10158, USA

Library of Congress Cataloging in Publication Data

Computer applications in water supply.
(Mechanical engineering research studies. Engineering control series ; 3, 4)
Proceedings of an international conference held at Leicester Polytechnic, in Sept. 1987.
Includes bibliographies and indexes.
Contents: v. 1. Systems analysis and simulation — v. 2. Systems optimization and control.
1. Water-supply engineering—Data processing—Congresses. 2. Water-supply engineering—Computer simulation—Congresses. 3. System analysis—Congresses. I. Coulbeck, Bryan, 1931– II. Orr, Chun-Hou, III. Series.

TC409.C65 1988 628.1'028'5 88-11475
ISBN 0 471 91783 4 (Wiley : v. 1)
ISBN 0 471 91784 2 (Wiley : v. 2)

British Library Cataloguing in Publication Data

Computer applications in water supply.
1. Water supply industries. Applications of computer systems
I. Coulbeck, Bryan, *1931–* II. Orr, Chun-Hou III. Series
338.4'76281'02854

ISBN 0-86380-062-9 V.1
ISBN 0-86380-063-7 V.2
ISBN 0-471-91783-4 V.1 Wiley

ISBN 0 86380 062 9 (Research Studies Press Ltd.)
ISBN 0 471 91783 4 (*John Wiley & Sons Inc.*)

Printed in Great Britain by SRP Ltd., Exeter

Contents