TECHNICAL PAPER
Journal of the South African Institution of Civil Engineering, 45(4) 2003, Pages 2–9 , Paper 548

Generic optimisation of small water reticulation networks using *Wadessy*

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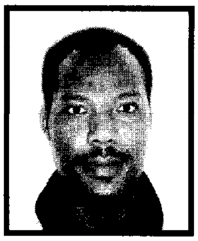

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The proliferation of water distribution system (WDS) optimisation algorithms has been extensive over the past four decades. The utilisation of these algorithms in practice has been less popular, as most models have been relegated to the confines of academia. The unsustainability of small water reticulation networks (WRNs) in many South African low-income communities continues to be a reality. This is particularly significant in light of the free basic water policy, which places the responsibility of basic potable water provision into the hands of local councils faced with the constraints of inadequate systems, sub-optimal systems, tight budgets and inadequate skills. To facilitate the provision of basic water to many poor communities, effective and efficient planning, design and operation tools are required. Presented herein is a generic and robust WRN design optimisation algorithm within a decision support system (DSS) software called Wadessy *(an acronym for WAter DEcision Support SYstem).* Wadessy *comprises a suite of computer programs useful for facilitating the optimal design, planning and daily operation of small WDSs. The design algorithm employed is based on the concept proposed by Featherstone and El-Jumaily (1983).*

The validity of Wadessy *is proved by comparison with a well-known example from Alperovits and Shamir's (1977) linear programming gradient (LPG) model and a practical case study in Selebi-Phikwe, Botswana.* Wadessy *achieved a 1,91 % cost saving in comparison to that obtained by Alperovits and Shamir (1977) and a 32,52 % cost saving (about 1 497 190 Pula) for the Selebi-Phikwe WRN, based on October 2001 pipe costs.*

Keywords: Wadessy*, design, optimisation, water reticulation network, cost*

LIST OF SYMBOLS

Q_i	pipe flow rate
h_f	pipe head loss
ΔE_{FGN}	difference in total hydraulic grade between fixed grade nodes (FGNs)
S_0	hydraulic gradient
d_i	pipe diameter
Q_K	pumping mains flow capacity
C	cost
t_i	pipe wall thickness
l_i	pipe length
λ	Darcy-Weisbach pipe friction factor
H_j	node pressure head
C_{HW}	Hazen-Williams pipe friction coefficient
g	acceleration due to gravity
NP	number of pipes in water reticulation network

INTRODUCTION

The design of water distribution systems (WDSs) has received a great deal of attention because of its importance to industrial growth and water's crucial role in society for health, fire-fighting, and quality of life, particularly in light of increased urban development and water use (Sherali *et al* 1998:1381). WRNs are essential components of all WDSs as they convey potable water from the source, pump station or storage to the consumers. The cost of these networks may amount to as much as 60 % of the entire water supply scheme (Sarbu & Borza 1997; Stephenson, 1998:49) and as a result, operation and maintenance costs may soar higher if networks are ill designed (Ilemobade & Stephenson 2002:80). WRNs also account for the largest costs in municipal maintenance budgets (Sherali *et al* 1998:1381). Despite often scarce resources, and particularly in light of the free basic water policy, all South African local governments are obligated to provide this resource. Since WRNs are composed mostly of pipes, pipe-sizing decisions have become critical in designing cost-effective WDSs that are capable of handling varied demand loadings and satisfying minimum pressure head requirements (Ilemobade & Stephenson 2002:80). Optimisation therefore provides an appropriate tool for achieving the best WRN designs.

Wadessy'S MODEL FORMULATION

The basic equations of continuity (1), conservation of energy (2) and pressure head difference (3) are utilised in modelling WRNs:

$$\Sigma(Q_{in} - Q_{out}) = 0 \qquad (1)$$

$$\Sigma h_f = \Delta E_{FGN} \qquad (2)$$

$$h_f = g(Q_i) \qquad (3)$$

Q_{in}, Q_{out} represent flows into and away from any node respectively; Σh_f is total energy loss around a loop; ΔE_{FGN} is difference in total hydraulic grade between fixed grade nodes (FGNs); and $g(Q_i)$ is pipe head loss equation as a function of flow, Q_i.

To arrive at an optimal solution, an iterative simulation-optimisation algorithm is employed. Efficient hydraulic simulation (both static and dynamic) is based on modelling the WRN using equations 1-3 and determining the unknown variables using the established Newton-Raphson iterative procedure (Haarhoff 1998) on simultaneous equations generated using the nodal method (Cornish 1939). Pipe sizes (which are initially assumed for new WRN designs) and other pipe parameters, consumer demands, network layout configuration, pump constants and FGN elevations are known prior to simulation. The Choleski decomposition technique (Stoer & Bulirsch 1993) is employed to generate the matrix for computing node residual pressure heads. Based on either the Darcy-Weisbach or Hazen-Williams pipe friction equations (14 and 15), continuity is checked at each network node, and if a violation exists, the entire simulation process is repeated. Output from the simulation include pipe flows and orientation, pipe headlosses, friction factors, node residual pressure heads, draw-off at each source node, pumping heads and valve head losses.

Wadessy's design optimisation procedure is adapted from Featherstone and El-Jumaily's (1983) model, which is based on the concept that a hypothetical linear hydraulic gradient, S_o, for a balanced WRN exists by which the initial network design can be iteratively corrected to produce optimal pipe sizes and an optimal relation between each pipe. In addition to Featherstone and El-Jumaily's (1983) model, the effects of hydraulic surfaces in determining optimal designs were previously undertaken by Deb and Sarker (1971), Wu (1975) and Alperovits and Shamir (1977).

Deb and Sarkar's (1971) study is limited to surfaces developed by rotating a parabola and straight line, between the inlet and the furthest point in the network, about a vertical axis through the inlet. The major drawbacks of this method are that the cost functions used are related to equivalent and not actual pipes, that A^1 is obtained from hypothetical flows, and that the pressure surface within the network is artificially created. A^1 is a constant for each network loop whose optimum value determines whether an appropriate network solution may be obtained ($A^1 = \Sigma[D^e{}_m / Q]$). $D^e{}_m$ represents equivalent pipe diameter, Q is pipe discharge in litres per minute and m is a constant derived from the pipe cost function). Wu (1975) showed that for a single pipe main composed of lengths of different diameters delivering water to the sub-mains in an irrigation system, the optimal shape of the energy gradient producing minimum cost of the pipeline is a curve at the middle section of the main with a sag of 15 % of the total head drop, below a straight line drawn between the inlet and outlet head elevations (figure 1). Fifteen energy gradient patterns, including concave and convex curves and a straight line, were imposed on the pipeline. The cost difference however, between the results of using the straight (linear) energy and the optimal (parabolic) energy gradient lines was found to be of the order of only 2 %. Alperovits and Shamir (1977) developed the linear programming gradient (LPG) method. At each iteration, an initial flow distribution is assumed for the network and this forms the basis for optimising each network pipe using linear programming. From the network design determined, the hydraulic gradient of the objective function is computed. Using this gradient value, the network's flow distribution and hence pipe sizes are revised. This two-stage procedure is iteratively performed until no further cost reduction occurs. The peculiar feature of this model lies in its ability to size major WDS components, to optimise the WDS under multiple loading conditions, and to determine optimal operating settings for pumps and valves under multiple loading conditions. Some weaknesses include the considerable skill required to set out and optimise a WDS since several heuristics are employed, and the need to try optimisation from several starting points if one is to be assured of not having missed a better design.

Objective function

Wadessy's optimisation objective emphasises the minimisation of capital and recurrent costs for the major components of a WDS and is mathematically modelled as follows:

$$\text{Minimise } C_{WDS} = (C_{WRN} + C_{pump\ and\ reservoir\ sub\text{-}system}) \qquad (4)$$

where

$$C_{pump\ and\ reservoir\ sub\text{-}system} = (C_{pump\ installation} + C_{pump\ operation} + C_{pumping\ mains} + C_{reservoir\ storage}) \qquad (5)$$

The minimisation of each major component is primarily a function of certain decision variables:

$$\text{Minimise } C_{WRN} = f(S_o, d_i) \qquad (6)$$

$$\text{Minimise } C_{pump\ and\ reservoir\ sub\text{-}system} = f(Q_k, d_i) \qquad (7)$$

The WDS objective function is subject to the following constraints:

- *Pipe diameter:*

$$d_{minimum} \leq d_i \leq d_{maximum} \qquad (8)$$

$$d_i \in (d_a, d_b, \ldots\ldots\ldots\ldots\ldots d_z)^* \qquad (9)$$

**set of all commercially available pipe sizes*

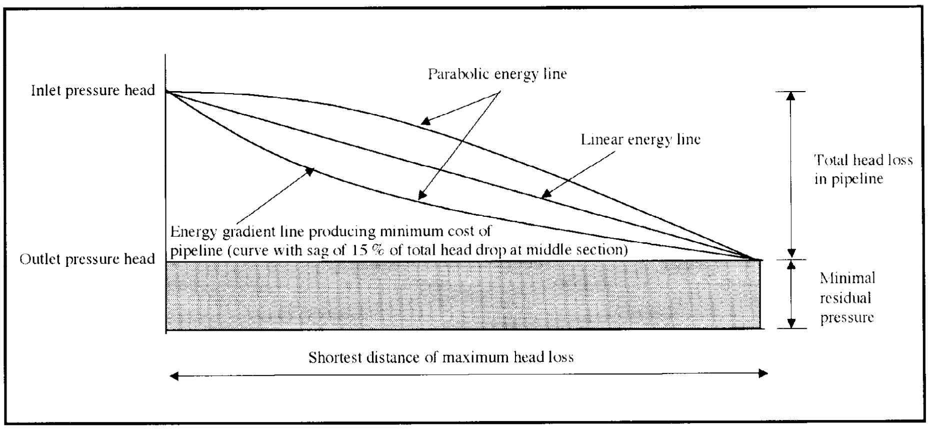


***Figure 1** Rotation of linear and parabolic energy lines about network inlet and outlet (after Featherstone & El-Jumaily 1983)*

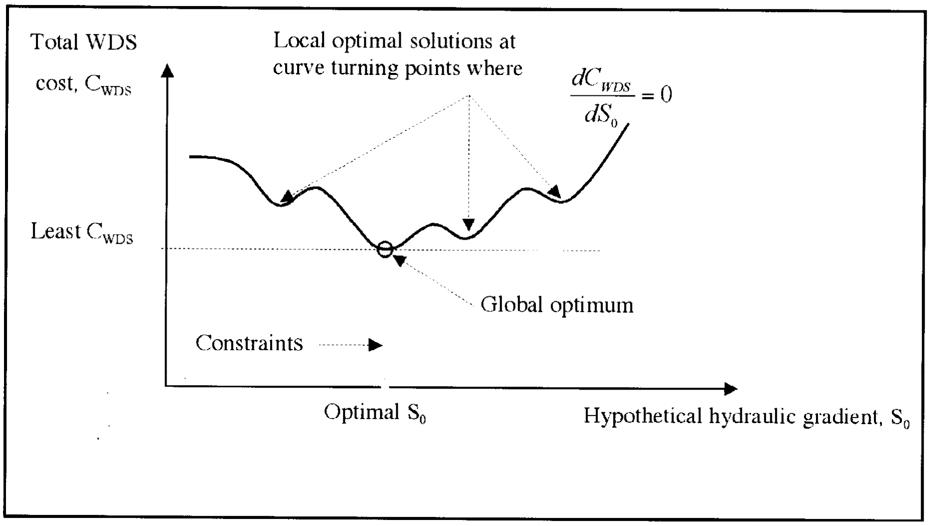


Figure 2 Non-linear variation of total system cost, C_{WDS} with hydraulic gradient, S_0

- *Pumping mains flow capacity:*

$$Q_{Average\ Hourly\ Demand} \leq Q_k \leq Q_{Maximum\ Hourly\ Demand} \qquad (10)$$

Q_k represents pumping mains flow capacity

- *Node pressure head:***

$$H_{minimum} \leq H_j \leq H_{maximum} \qquad (11)$$

***$H_{minimum}$ and $H_{maximum}$ represent the minimum and maximum allowable residual pressure heads for any node j*

- *Minimum system cost:****

$$C_{WDS\ minimum} \leq C_{WDS\ previous} \qquad (12)$$

C represents cost

****At each run, the non-linear optimisation procedure follows a concave relationship at each section of the curve (figure 2). The optimisation procedure thus determines the local optimum cost solution nearest to its starting point. When a local optimum is reached, the optimisation procedure will terminate. Several runs are recommended before finalising on a global optimum solution.*

- *Minimum variables:*

$$d_i,\ l_i,\ t_i \geq 0 \qquad (13)$$

S_o represents hydraulic gradient; d_i, pipe diameter; l_i, pipe length; and t_i, pipe wall thickness

The optimisation process

The Darcy-Weisbach (14) and Hazen-Williams (15) headloss equations are presented below,

$$d_i = \left(\frac{8\lambda l_i Q_i^2}{g\pi^2 h_f}\right)^{0,20} \qquad (14)$$

$$d_i = \left(\frac{10,7 l_i Q_i^{1.85}}{C_{HW}^{1.85} h_f}\right)^{0,21} \qquad (15)$$

λ represents the Darcy-Weisbach pipe friction factor and C_{HW}, the Hazen-Williams pipe friction coefficient. WRN pipe costs may be represented by the generic equation below (Barta & Rowse 1998):

$$C_{WRN} = \sum_{i=1}^{NP}\left[b_1 l_i^{b2} d_i^{b3} t_i^{b4}\right] \qquad (16)$$

where *b1, b2, b3* and *b4* are pipe cost variables. By substituting d_i in equation 14 or 15 into equation 16, and equation 16 into equation 4, the objective function for the optimisation process becomes

$$C_{WDS} = \sum_{i=1}^{NP}\left[b_1 l_i^{b_2}\left(\frac{R_3 Q_i^{R_1}}{S_0^{R_2}}\right)^{b_3} t_i^{b_4}\right] + C_{pump\&reservoir-subsystem} \qquad (17)$$

for the Darcy-Weisbach equation, $R_1 = 0,40;\ R_2 = 0,20;\ R_3 = 0,61\lambda^{R_2}$
for the Hazen-Williams equation, $R_1 = 0,38;\ R_2 = 0,21;\ R_3 = C_{HW}^{-R_1}$

Equation 18 results from re-arranging equation 17 with respect to S_0:

$$S_0 = \left(\frac{\sum_{i=1}^{NP}\left[b_1 l_i^{b_2}\left(R_3 Q_i^{R_1}\right)^{b_3} t_i^{b_4}\right]}{C_{WDS} - C_{pump\&reservoir-subsystem}}\right)^{\frac{1}{R_2 b_3}} \qquad (18)$$

The dummy hydraulic gradient, S_0, thus becomes the variable that iteratively corrects WRN pipe diameters, d_i, until an optimal solution is reached. Once S_0 is calculated, its value is substituted into the appropriate head loss equation (14 or 15) to correct previous pipe sizes. Simulation process is then performed on the WRN comprising the newly corrected pipe sizes

Table 1 Node data

Node ref	Elevation (m)	Node minimum residual pressure head (m)	Consumption (m³/h)	
			Peak flow	Night flow
1	210	0	-420,00**	-300,00**
2	150	30	100,00	0,00
3	160	30	100,00	0,00
4	155	30	120,00	0,00
5	150	30	270,00	0,00
6	165	30	330,00	0,00
7	160	30	200,00	0,00
8	195	0	-700,00**	300,00

*****Negative consumption represents supply to the node (ie source node)***

Table 2 Pipe data

Pipe ref	Length (m)	C_{HW}	Range of allowable diameters (in)
1	1 000	130	0–20
2	1 000	130	0–20
3	1 000	130	0–20
4	1 000	130	0–20
5	1 000	130	0–20
6	1 000	130	0–20
7	1 000	130	0–20
8	1 000	130	0–20
9	100	130	0–20

Table 3 Pipe cost data

Pipe diameter (in)	1	2	3	4	6	8	10	12	14	16	18	20
Unit cost per m ($)	2	5	8	11	16	24	32	50	60	90	130	170

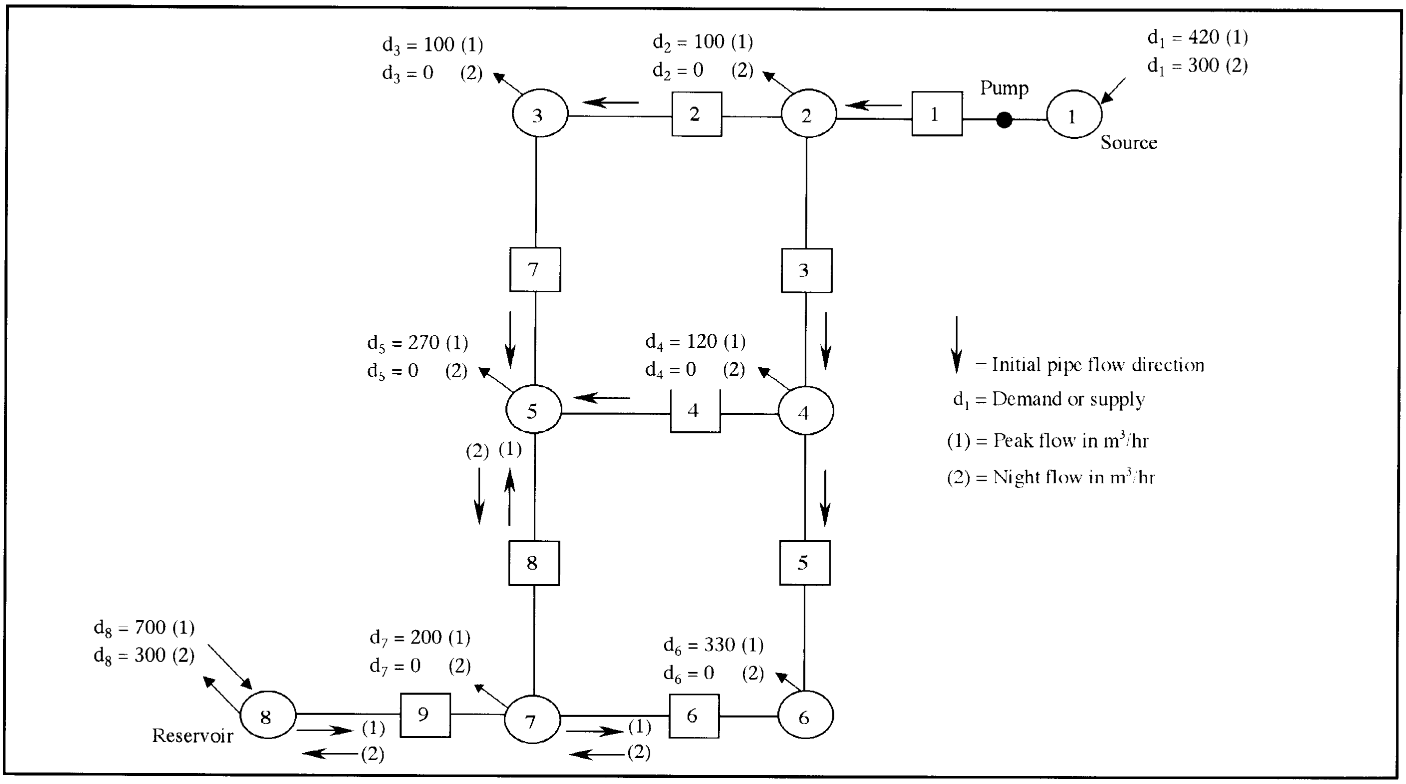


Figure 3 Example network

Table 4 Comparison of pipe optimisation output for figure 3 from three models

Pipe ref	Start node	End node	Alperovits and Shamir (1977) (segmental links)						Farmani *et al* (1999) (discrete links)		*Wadessy* (discrete links)					
			Length 1++ (m)	Dia (in)	Length 2++ (m)	Dia (in)	Peak flow Head loss (m)	Night flow Head loss (m)	Length (m)	Dia (in)	Length (m)	Dia (in)	Peak flow Flow, (m³/hr) +	Peak flow Head loss (m)	Night flow Flow (m³/hr)+	Night flow Head loss (m)
1	1	2	33,89	10,00	966,08	12,00	8,99	4,82	1 000	12,00	1 000	14,00	748,08	13,77	486,00	6,20
2	2	3	374,46	5,00	625,51	8,00	10,91	3,12	1 000	8,00	1 000	8,00	214,56	22,60	173,16	15,21
3	2	4	1 000,00	10,00	0,00	0,00	5,82	7,65	1 000	10,00	1 000	12,00	433,44	10,87	312,84	5,94
4	4	5	816,14	6,00	183,85	8,00	5,39	3,96	1 000	1,00	1 000	1,00	0,72	18,81	0,72	24,49
5	4	6	999,97	10,00	0,00	0,00	0,19	5,08	1 000	8,00	1 000	10,00	312,84	14,84	312,12	14,75
6	6	7	999,99	14,00	0,00	0,00	2,12	0,99	1 000	12,00	1 000	10,00	-17,28	0,07	312,12	14,75
7	3	5	929,62	6,00	70,36	4,00	0,30	8,49	1 000	10,00	1 000	8,00	114,48	7,07	173,16	15,21
8	7	5	825,67	10,00	174,28	8,00	7,32	2,11	1 000	12,00	1 000	10,00	154,80	4,04	-173,88	5,01
9	7	8	100,00	16,00	0,00	0,00	0,54	0,11	100	14,00	100	14,00	-372,24	0,31	486,00	0,51
WRN Pipe Cost			$267 113,00						$268 000,00		$262 000,00					

+Negative flow represents flow in the direction opposite to the initial flow orientation

++Total length of pipe link between two nodes is the sum of length 1 plus length 2

Table 5 Comparison of node optimised output for figure 3

Node ref	Minimum residual pressure head (m)	Alperovits and Shamir (1977) Pressure head (m) Peak flow	Alperovits and Shamir (1977) Pressure head (m) Night flow	*Wadessy* Pressure head (m) Peak flow	*Wadessy* Pressure head (m) Night flow
1	0,00	0,00	N/A	0,00	0,00
2	30,00	N/A	N/A	70,33	77,90
3	30,00	30,10	N/A	37,73	52,69
4	30,00	40,20	N/A	54,46	66,96
5	30,00	39,80	N/A	40,66	47,48
6	30,00	30,00	N/A	29,62	42,21
7	30,00	37,10	N/A	34,69	32,47
8	0,00	0,00	N/A	0,00	0,00

Farmani et al (1999) did not publish pressure head values for their optimised WRN for figure 3

to calculate new node pressure heads. This iterative procedure is performed until one or more constraints (equations 8-13) are violated. A separate methodology is used to calculate $C_{pump\&reservoir\text{-}subsystem}$ (the optimal value is simply inserted into equation 18 if available).

MODEL VALIDATION

In validating the LPG model, Alperovits and Shamir (1977) present an example network shown in figure 3. The operation of the network is studied under peak and night flow conditions. Basic network data

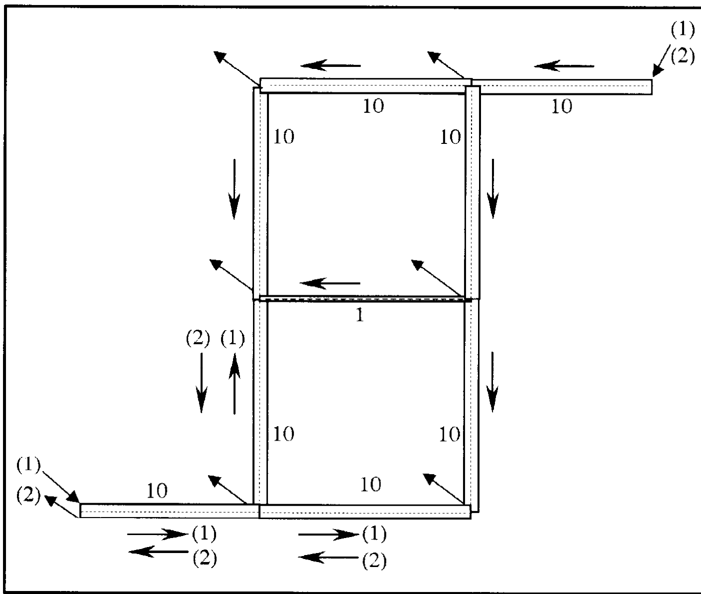


Figure 4a Initial pipe flow and network design

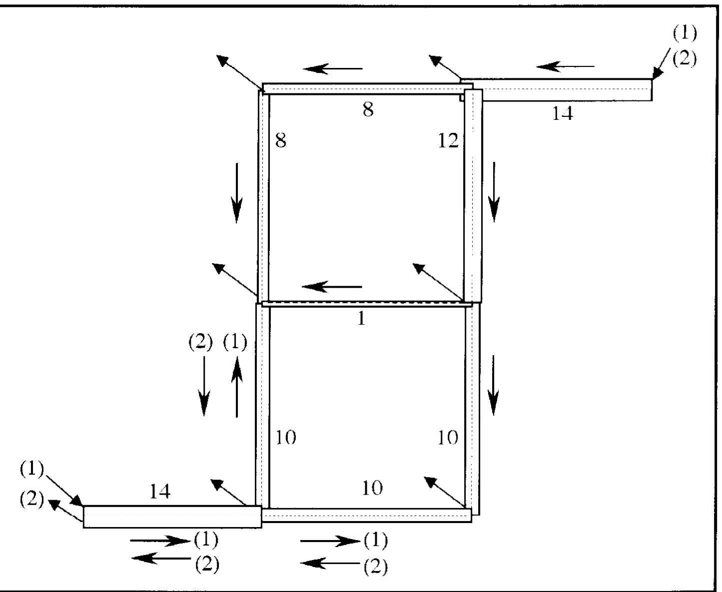


Figure 4b Optimal pipe flow and network design

= Pipe flow direction

= Node draw-off or supply

10 = Pipe diameter (in)

(1) = Peak flow in m³/hr

(2) = Night flow in m³/hr

is given in tables 1 and 2 respectively, while pipe costs, given in arbitrary units, are as shown in table 3 (Alperovits & Shamir 1977:892). Minimum permissible pressure heads for each demand node is 30 m. Since several workers have used the results presented by Alperovits and Shamir (1977) to validate their models, a comparison of the results obtained by Alperovits and Shamir (1977), Farmani *et al* (1999) and *Wadessy* are presented in tables 4 and 5.

The optimal solution achieved by *Wadessy* was obtained after running the optimisation procedure from several starting network designs. A 1,5 % deviation from the minimum residual pressure (30 m) was permitted during computations, hence a computed residual pressure head of 29,62 m for Node 6 (the critical node). The global optimum network design cost computed by *Wadessy* is $262 000,00 compared to $267 113,00 and $268 000,00 computed by Alperovits and Shamir (1977) and Farmani *et al* (1999) respectively. *Wadessy* thus achieves a cost saving of 1,91 % from Alperovits and Shamir's (1977) solution and 2,24 % from Farmani *et al*'s (1999) solution. The cost savings achieved by *Wadessy* is despite the fact that *Wadessy* provides only one pipe size per link (discrete links), in contrast to Alperovits and Shamir (1977), who provide multiple pipe sizes per link (segmental links). Irrespective of the magnitude of starting diameters, the algorithm searches for the best pipe network design closest to it (figure 2). Schematics of the initial and optimal designs are presented in figures

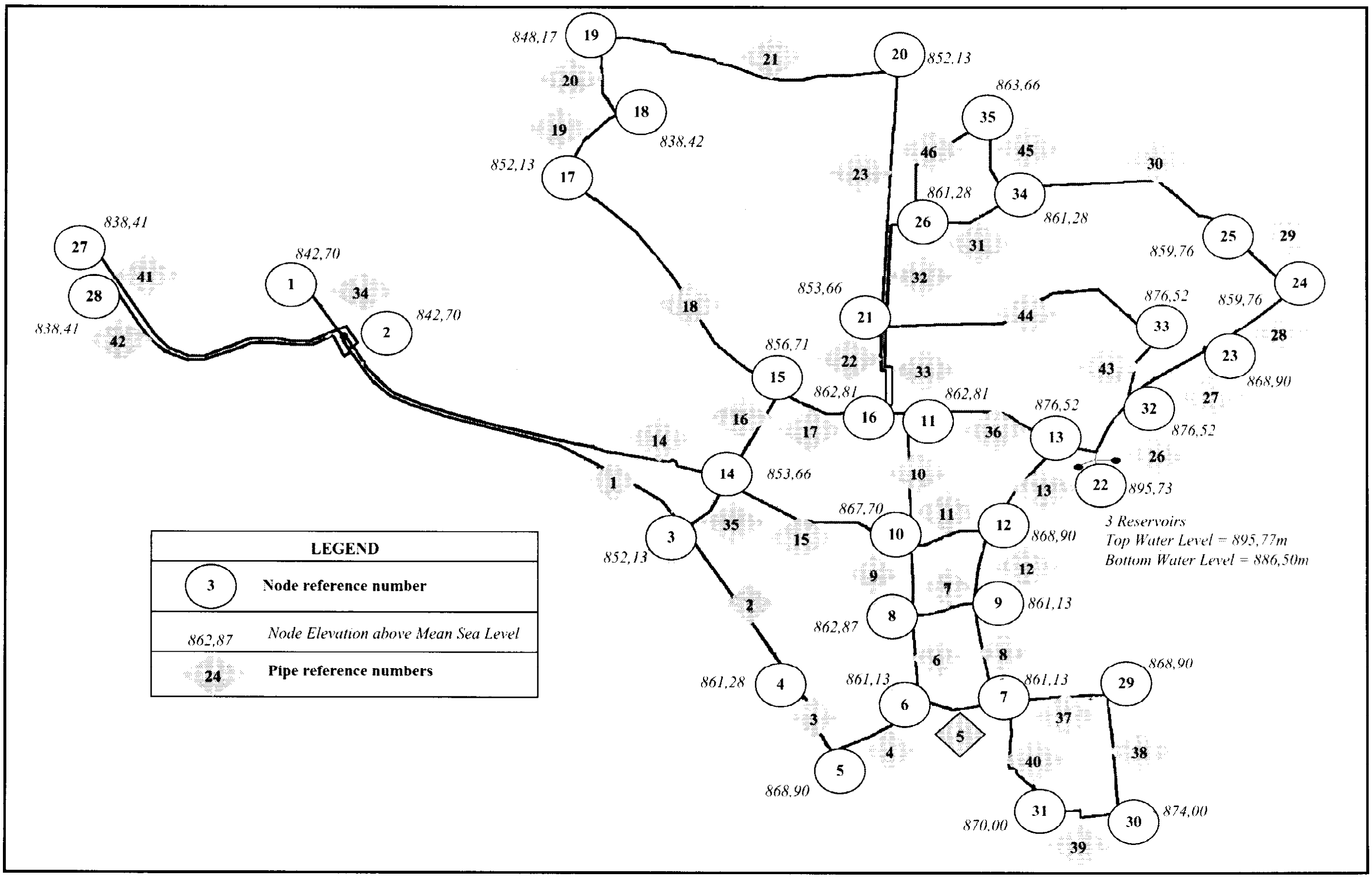


Figure 5 Existing Selebi-Phikwe water reticulation network (skeletonised)

4a and 4b. The tendency towards a branching network can be seen as distance from each FGN increases and as demand at a node, especially during peak flow, is supplied from both sources. Until an optimal solution is reached, flow distribution is treated as a variable.

In a practical network of this magnitude, a 1-inch pipe, as recommended for Pipe 4, would be meaningless. By replacing with a 2-inch pipe, the WRN costs \$265 000,00: a saving of 0,79 % and 1,12 % from the solutions presented by Alperovits and Shamir (1977) and Farmani *et al* (1999). Changes in residual pressures and flows (table 4) due to this replacement are insignificant: a pressure of 29,61 m was computed for the critical node (Node 6).

Case study: Selebi-Phikwe, Botswana

Wadessy was employed to analyse the existing Selebi-Phikwe WRN in order to determine the performance of the existing network (figure 5 and table 6) on the addition of three new residential developments. During the analysis, *Wadessy* was also used to hypothetically determine the optimal pipe network design for Selebi-Phikwe in comparison to that existing, based on October 2001 consumer demands and pipe costs (table 7). Average peak and night flows calculated for October 2001 were 0,270 m^3/s and 0,135 m^3/s respectively. 20 M*l* of storage is currently provided in three concrete cylindrical tanks situated at the southeast section of Selebi-Phikwe. Minimum and maximum permissible node pressure heads are 15 m and 90 m respectively (WUC 1995). Minimum pipe sizes connecting users to the reticulated mains and on which fire hydrants are expected are 63 mm and 75 mm respectively (WUC 1995). Three pumps are installed in the existing pumpstation; one of which acts as standby. The polynomial equation: $H_p = AQ^2 + BQ + C$ adequately represents pumping head within the WRN. For two pumps in parallel: A = -414,94; B = -3,50 and C = 88,93, and for pumps operating individually: A = -1 660,30; B = -12,31 and C = 90,04.

Detailed results of the optimisation are presented in figures 6, 7 and 8. Figures 7 and 8 present node residual pressure head results for the existing and *Wadessy*-optimised WRN, based on peak and night flow conditions. It can easily be seen that although both network designs are adequate under night flows,

Table 6 Selebi-Phikwe WRN pipe and node data

Pipe ref	Length (m)	Diameter (mm)	Pipe material*	Node ref	Elevation (m)	Consumption (m^3/hr)		Min residual pressure head (m)
						Peak flow	Night flow	
1	2 950,00	300,00	AC	1	842,70	-705,96	-486,00	0,00
2	1 800,00	110,00	uPVC	2	842,70	0,00	0,00	15,00
3	310,00	100,00	AC	3	852,13	0,00	0,00	15,00
4	850,00	200,00	uPVC	4	868,90	89,42	0,00	15,00
5	600,00	200,00	uPVC	5	868,90	89,42	0,00	15,00
6	650,00	110,00	uPVC	6	876,52	0,00	0,00	15,00
7	450,00	250,00	uPVC	7	861,13	0,00	0,00	15,00
8	800,00	250,00	uPVC	8	862,81	39,37	0,00	15,00
9	550,00	110,00	uPVC	9	861,13	0,00	0,00	15,00
10	1 000,00	200,00	uPVC	10	856,71	44,71	0,00	15,00
11	680,00	300,00	AC	11	862,81	77,76	0,00	15,00
12	560,00	250,00	uPVC	12	868,90	48,60	0,00	15,00
13	880,00	300,00	AC	13	876,52	73,87	0,00	15,00
14	4 050,00	300,00	AC	14	853,66	40,82	0,00	15,00
15	1 470,00	300,00	AC	15	856,71	32,04	0,00	15,00
16	800,00	300,00	AC	16	862,81	7,78	0,00	15,00
17	780,00	300,00	AC	17	852,13	36,00	0,00	15,00
18	2 450,00	200,00	AC	18	838,42	0,00	0,00	15,00
19	1 080,00	200,00	uPVC	19	848,17	93,31	0,00	15,00
20	250,00	160,00	uPVC	20	852,13	55,40	0,00	15,00
21	2 330,00	160,00	uPVC	21	853,66	0,00	0,00	15,00
22	800,00	200,00	AC	22	895,73	-266,04	486,00	0,00
23	2 000,00	200,00	uPVC	23	868,90	0,00	0,00	15,00
24	200,00	300,00	AC	24	859,76	0,00	0,00	15,00
25	180,00	450,00	AC	25	859,76	5,22	0,00	15,00
26	600,00	500,00	AC	26	861,28	19,19	0,00	15,00
27	680,00	500,00	AC	27	838,41	18,47	0,00	15,00
28	930,00	450,00	AC	28	838,41	28,80	0,00	15,00
29	460,00	350,00	AC	29	868,92	16,52	0,00	15,00
30	1 560,00	300,00	AC	30	874,00	0,00	0,00	15,00
31	740,00	300,00	AC	31	870,00	16,52	0,00	15,00
32	1 000,00	300,00	AC	32	876,52	0,00	0,00	15,00
33	800,00	300,00	uPVC	33	876,52	35,57	0,00	15,00
34	20,00	400,00	AC	34	861,28	38,39	0,00	15,00
35	540,00	300,00	AC	35	861,28	64,80	0,00	15,00
36	1 170,00	300,00	AC					
37	820,00	160,00	uPVC					
38	950,00	110,00	uPVC					
39	700,00	100,00	uPVC					
40	800,00	75,00	uPVC					
41	2 420,00	75,00	uPVC					
42	2 180,00	75,00	uPVC					
43	510,00	160,00	uPVC					
44	2 270,00	160,00	uPVC					
45	510,00	200,00	AC					
46	1 085,00	200,00	AC					

AC and uPVC represent asbestos cement and unplasticised polyvinyl chloride pipes respectively

Table 7 Commercially available pipe sizes and costs for Selebi-Phikwe (October 2001)

	Dia (mm)	63	75	90	100	110	150	160	200	250	300	350	400	450	500
AC	Class	-	12+	-	12	12+	12+	-	12	-	12	12+	12+	12+	12+
	Cost/m[a]	-	11,4*	-	15,8	25,7*	27,4	-	29,4	-	161,0	183,0	200,0*	220,0*	230,0*
uPVC	Class	9	9	9	9	9	-	9	9	6	9+	-	-	-	-
	Cost/m[a]	8,0	11,4	2,4	7,2	25,7	-	45,5	43,1	169,0	161,0	-	-	-	-

[a]Pipe costs are in Botswana pula; +class of pipe assumed; *pipe cost per metre assumed as actual cost not available; AC represents asbestos cement pipes, while uPVC represents unplasticised polyvinyl chloride pipes

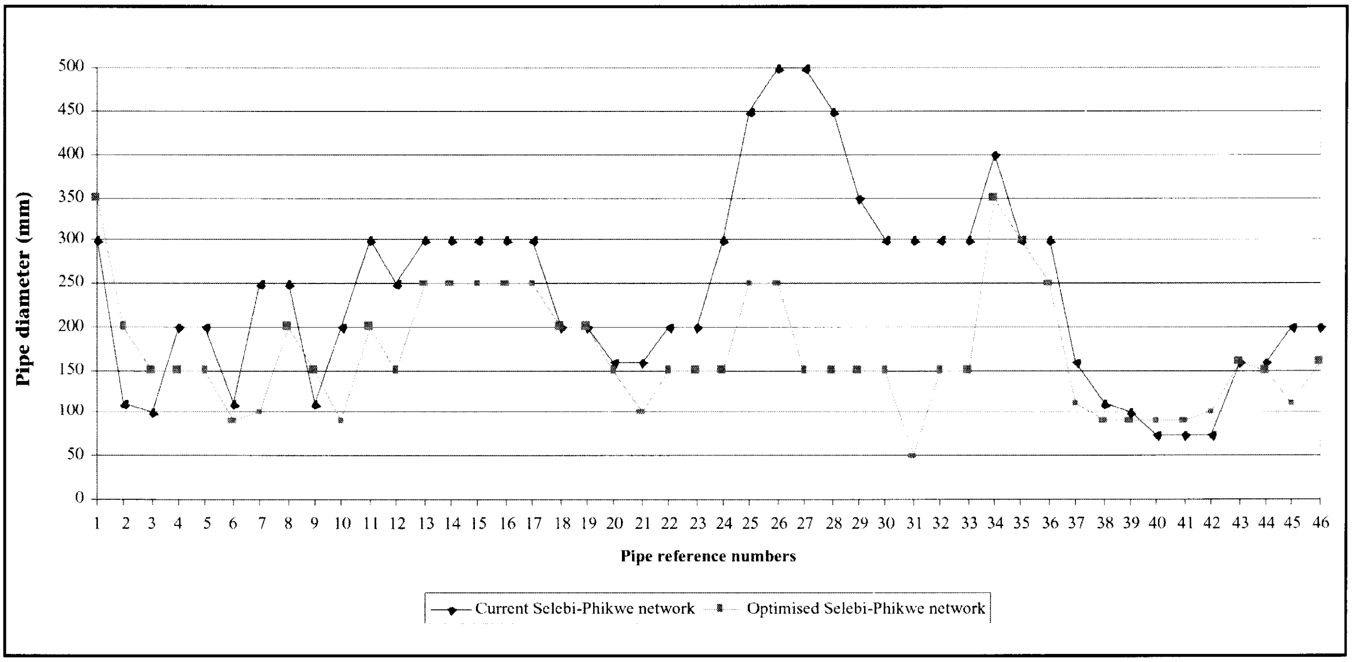


Figure 6 Existing and Wadessy-optimised Selebi-Phikwe pipe reticulation network

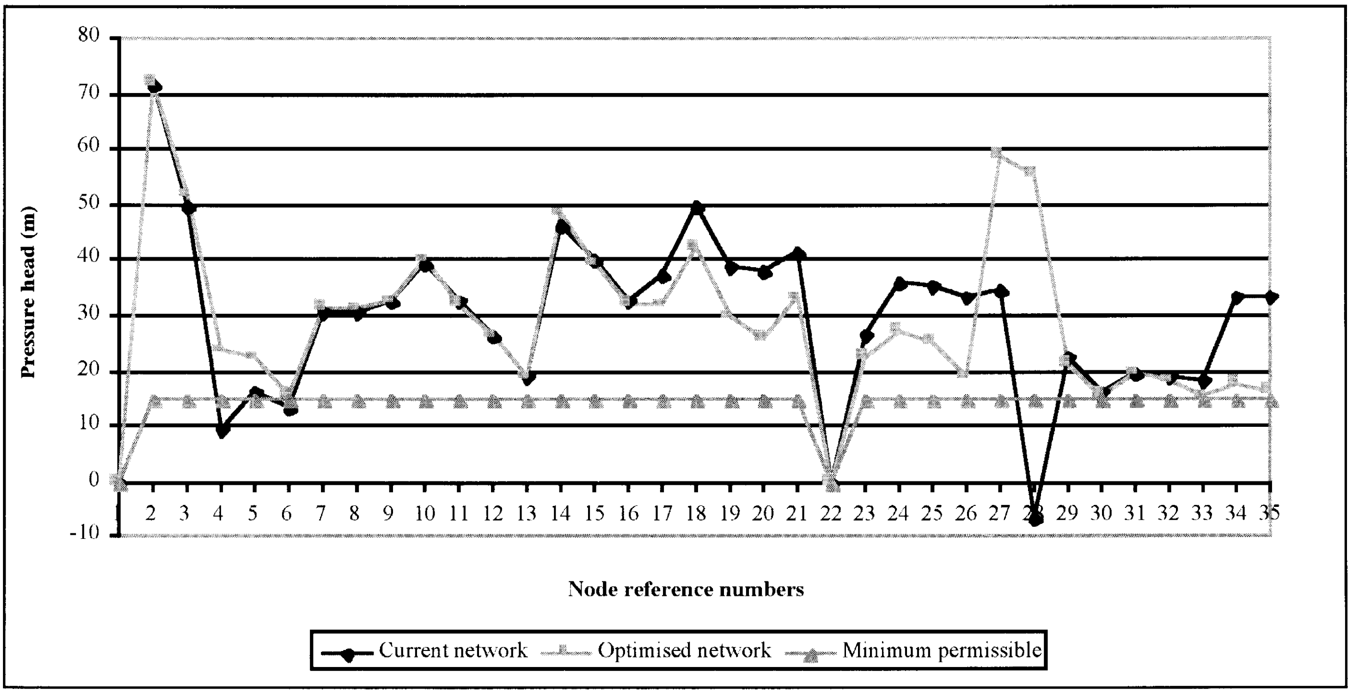


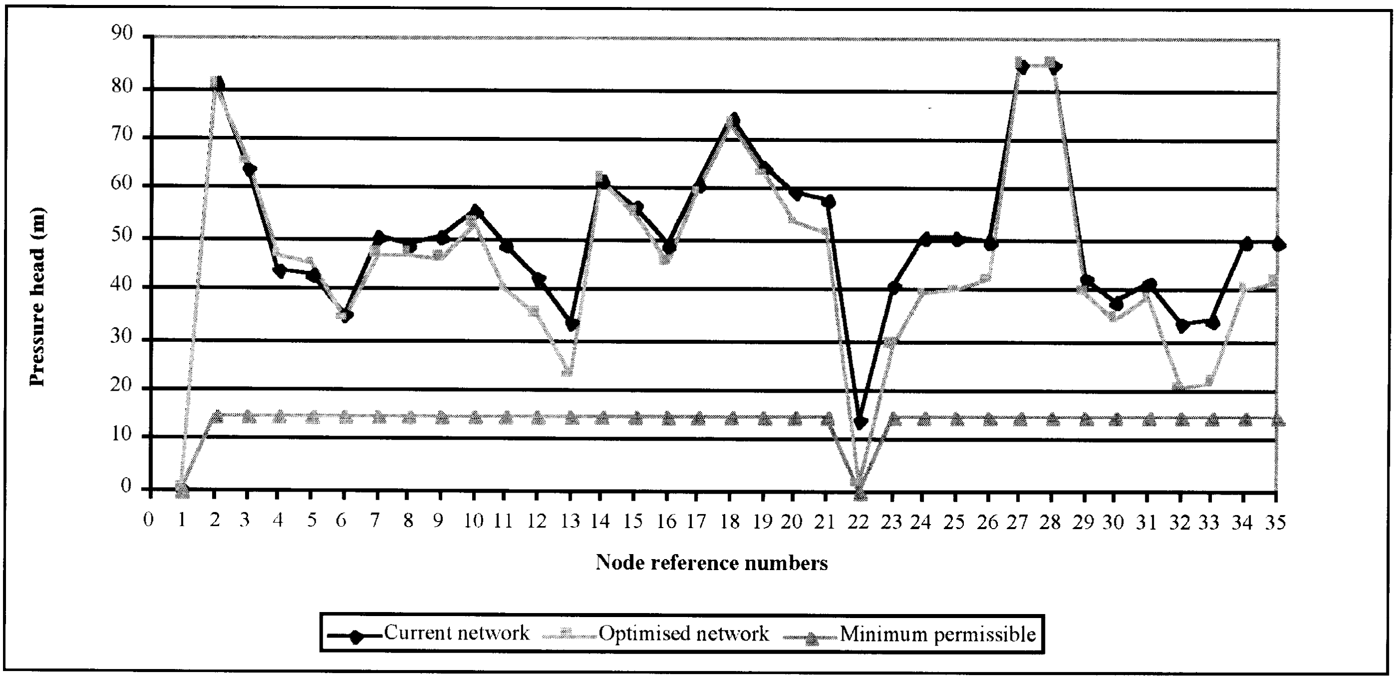
Figure 7 Existing and Wadessy-optimised node pressure head results for peak flows

the existing network design is deficient under peak flows where the specified minimum permissible nodes pressure heads (15 m) are violated in nodes 4, 6 and 28 (9,64 m, 13,42 m and -6,64 m respectively) – see figure 7. This is because pipes 2 and 42 in the existing Selebi-Phikwe WRN (figure 6) generate high headlosses (26,44 m and 82,84 m respectively) during peak flows, as their capacities are small (110 mm and 75 mm respectively) in relation to the high demands at their downstream nodes (nodes 4, 6 and 28). Pipe 42 supplies the Botswana Defence Force camp, which currently experiences low or no flow during peak demand periods. Better flow conveyance and reduced headlosses in pipe 42 is enhanced in *Wadessy's* optimal design by the provision of a 100 mm uPVC pipe.

Based on October 2001 prices, it is estimated that 1 497 190,00 Pula (about 32,52 %) could have been saved from the 4 603 914,00 Pula estimated to have been spent on the installation of the existing pipe network alone.

CONCLUSIONS

Wadessy utilises efficient algorithms to facilitate effective decision-making in the planning, design, analysis and operation of small WDSs. In this paper, *Wadessy's* WRN design algorithm is validated by applying it to a popular design example first proposed by Alperovits and Shamir (1977), and to the existing Selebi-Phikwe WRN. By permitting a 1,5 % deviation from the minimum node pressure head of 30 m, a 1,91 % cost saving was achieved using *Wadessy's* design algorithm in comparison to that determined by Alperovits and Shamir (1977). A 32,52 % cost saving (1 497 190,00 Pula) was also achieved by using *Wadessy's* design algorithm in comparison to the cost of the existing Selebi-Phikwe WRN based on October 2001 prices. By minimising the changes in node pressure heads during optimisation, the optimal



interaction between each network pipe was enhanced.

SOFTWARE PROCUREMENT

Wadessy's WRN design software may be obtained from A A Ilemobade (Tel: 011-717-7153; Fax: 011-339-1762; e-mail: Adesola@civil.wits.ac.za) at a minimal fee.

Acknowledgement

The authors are indebted to the Bradlow Foundation and Richard Ward Endowment Fund (fund no TW CIVN ILEM) for funding this research.

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