WATER DISTRIBUTION NETWORK MODELLING OF UNIVERSITY OF PORT HARCOURT THREE CAMPUSES

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ABSTRACT

The University of Port Harcourt (Uniport) has three campuses, all within 1km radius and each maintains an independent water distribution network. The study explored the possibility of an integrated water distribution network by computer-aided simulation. The software adopted for the simulation is the popular WASDIM developed based on the Hardy-Cross relaxation method. Seven simulation options were advanced, the first three cases are when the overhead tank (OHT) in each of the campus is allowed to function alone; the next three cases are when we permit a combination of two OHTs functioning simultaneously (while the 3rd OHT is shutdown) and the last case is when the three OHTs are functioning simultaneously. The computed heads at various nodes (27 in No.) at the end of 200 iterations for each simulation option are compared and contrasted with respect to the option producing the best performance (the simulation options (1 thro 7) producing the highest pressure head(s)). On the basis of frequency distribution as per best performance, the following ranking (1st to 7th positions) was obtained: Option 4, Option 1, Option 2, Option 7, Option 5, Option 6 and Option 3, respectively. Results also indicate that the pipe material used for water mains do significantly affect the pressure head distribution in the network. Similarly, pressure responses due to stepwise increase in OHT elevation followed the hydrostatic pressure variation and the maximum pressure head development varies with the type of pipe material used.

Key words: *Water distribution network, WASDIM Software, Hardy-Cross Relaxation method.*

1.0 INTRODUCTION

Many methods are reported in literature dealing with water distribution network analysis. The "Hardy-Cross method" proposed by Cross (1936) is a trial and error (relaxation) solution approach to the Darcy-Weisbach formula for headloss computation and Von-Karman's friction factor formula for turbulent flow. Corrections are applied to the assumed pressure heads or flow rates until an acceptable hydraulic balance is achieved. Because the procedure involves a number of trials, convergence depends on the accuracy of the heads or flow rates originally selected (Nwaogazie and Okoye, 1994). Martin and Peters (1963) described the Newton-Ralphson iteration method to solve water distribution system problems. This method had much improved convergence characteristics and forms the basis for more general applications (Jeppson, 1977). The Newton-Ralphson, has the following disadvantages: i) Its convergence is rapid if the initial guessed values are chosen very close to the actual values and ii) It requires the Jacobian matrix (partial derivations), and its inversion at each iteration. Besides, the Jacobian matrix has to be updated at each iteration.

Wood and Charles (1972), Lewis and Kelvin (1980) introduced and adapted the linear theory to pipe network analysis. This method solves the entire set of hydraulic equations simultaneously after linearizing the non-linear terms in the energy equation. The linearization method has the advantage of ease of understanding, programming simplicity and flexibility in the representation of pumps, unfortunately this method is only suitable for simple networks (Mays, 2001).

The Hardy Cross method has the advantage over other methods in computer memory economy as it does not involve the solution of large sets of linear simultaneous equations. Among the widely used softwares for water supply network analysis is WASDIM programme (Nwaogazie and Okoye, 1994). The software is constructed using the principles of Hardy Cross method as a solution approach to the Darcy-Weisbach headloss equation and Von-Karman's friction formula for turbulent flow. The WASDIM is very flexible programme with provisions for both closed loop and branching systems. The WASDIM software was adopted for the present study on University of Port Harcourt integrated water supply scheme.

2.0 THE HARDY-CROSS METHOD

The relationship between headloss and velocity in a pipe is expressed by the Darcy -Weisbach equation:

$$h_L = f\frac{L}{D}\frac{V^2}{2g} \text{ - - - - - - - - - } (1)$$

In which h_L is head loss, L is Length, D is diameter, V is mean velocity, g is acceleration due to gravity and f is friction factor.

Equation (1) is dimensionally balanced and for computer application, f can be evaluated from the Colebrook-White formula:

$$\frac{1}{\sqrt{f}} = 2.\log\left(\frac{e}{3.7D} + \frac{2.52}{R\sqrt{f}}\right) \text{ - - - - - - - - } (2)$$

In which e is pipe roughness and R is Reynolds number

Or by the Von-Karman's friction formula:

$$\frac{1}{\sqrt{f}} = 2\log\frac{D}{e} + 1.14 \qquad \text{---------(3)}$$

For fully turbulent flows in pipe lines, we combine Equations (1) and (3), as:

$$Q = [\frac{\Pi}{4}\sqrt{2g}\,(2Log\frac{D}{e} + 1.14)D^{2.5}](\frac{h_L}{L})^{1/2} \quad \text{--- (4)}$$

Or

$$Q = K\,(\frac{h_L}{L})^{1/2} = K\sqrt{S} \qquad \text{---- (5)}$$

Where K is Conveyance, S is slope of the energy gradeline.

In the network analysis using the Hardy-Cross method, the first guess value for flow in the network is undoubtedly in error; therefore a correction in discharge, ΔQ, will have to be applied to satisfy the headloss requirement. Apparently, the error, ΔQ must be adjusted by an amount ΔH. The aggregate effect of ΔH on the excess Q at a junction may be evaluated by the weighted formula (Simon, 1981; Nwaogazie and Okoye, 1994):

$$\Delta H = \frac{2\Delta Q}{\sum|\,Q/h_L\,|} \qquad \text{------ (6)}$$

in which the denominator is the sum of all incoming and outgoing discharges divided by their respective head losses in the connecting pipes.

The WASDIM Software is constructed using Equations (4 and 6) in an iterative fashion based on the principles of Hardy-Cross relaxation method discussed earlier. The program consists of three sub-programs: Main program and two subroutines (Nwaogazie and Okoye, 1994). The same software was adopted for the simulation of the Uniport water supply network.

3.0 APPLICATION TO UNIPORT WATI SUPPLY DISTRIBUTION NETWORK

3.1 Water Distribution Network and Simulation Options

The study area consists of University of Port Harcourt three campuses, namely Choba Park, Delta Park and University Park (Unipark) which includes staff residential areas (Ghana-Ama, Gambia-Ama and University Teaching Hospital) see Fig. 1. As at the time of this study, the existing water supply distribution network is localized for each campus area. The present study is to integrate the water distribution network for the three campuses with five close loops and five dead ends (see Fig. 2). The network is arranged into thirty-one (31) pipe lengths and twenty seven (27) nodal points. The network is fed by three service reservoirs (overhead water storage tanks) situated at Unipark, Delta Park and Choba Park, respectively. On the characteristics of each pumping station, we have the following: (1) Unipark station with 2Nos. 20 Horse power (HP) pumps with a rated pump capacity of $25m^3/hr$ each; (2) Delta Park Station with 1 No. 20 HP and 1 No. 15 HP pumps with a rated capacities of $18m^3/hr$ and $15m^3/hr$, respectively; and (3) Choba Park station with 2 Nos. 20 HP pumps each with same capacity as those of Unipark station. For each pumping station one pump is usually on standby while the second pump is operational.

The various simulation options representing different pressure head scenarios and overhead tanks combinations are: (1) Unipark pumping station running alone; 2) Delta Park pumping station running alone; 3) Choba Park pumping station running alone; 4) Unipark and Delta Park pumping stations running simultaneously; 5) Unipark and Choba Park pumping stations running simultaneously; 6) Delta Park and Choba Park pumping stations running simultaneously and; 7) Unipark, Delta Park and Choba Park pumping stations running simultaneously. Seven input data files representing each option were designed and run for the above cases one after the other. For a particular option, the overhead tank (OHT) operating or the combination of OHTs is or are assigned the highest initial pressure heads. The initial guess values for pressure head for each node was made in accordance with the heights of respective overhead tank(s) and its expected decay distribution. A typical case of option-4 is adopted as illustrative example.

3.2 Results and Discussion

The results of option-4 are presented in details on account of it being the best option while others are summarized. For option 4, we have a simultaneous operation of Unipark (UPPS) and Delta Park Pumping Station (DPPS) while the Choba Park Pumping Station (CPPS) was shotdown. Consequently, the arrows depicting the flow directions originated from UPPS and DPPS as a combined effect. Thus, UPPS and DPPS have the highest initial pressure head values. The guessed flow network diagram is as shown in Fig. 2, while the results of the network simulation after 200 iterations and accumulated error on ΔH are shown in Figs. 3 and 4, respectively. The final flow direction (Fig. 3) was determined using the results after 200 iterations.

From Fig. 4, the accumulated error on ΔH after the first iteration is 1.52m, and decayed sharply to 0.382m after 11 iterations, then decayed gradually to 0.0903m after 61 iterations, to 0.0736m after 71 iterations, to 0.0667m after 81 iterations, to 0.0813m after 91 iterations, respectively. Thereafter, it decayed asymptotically to about 0.030m till after 200 iterations. Thus after 200 iterations, all OHTs combinations approached the true solution and the accumulated error on ΔH became stable. The first guesses affected the initial decay rates, but they all stabilized after 200 iterations. The percentage decrease on accumulated error on ΔH after 11 iterations is 74.9%, after 21 iterations 43.5% and decayed gradually to 11.4% after 61 iterations, and about 7% till after 200 iterations. By graphical assessment of the decay characteristics of Fig. 4, the errors at each stage of the computation are not amplified as the computation progresses.

The results of the other options (1-3 and 5-7) are briefly summarized:

OPTION 1 (UPPS running alone) produced the second best in terms of pressure rating. It also produced the highest pressures in fourteen nodes (Table 1). It has considerable influence in the pressure development up to and including Ali Cape Verde (staff quarters), see Fig. 6.

OPTION 2 (DPPS running alone) produced the third best overall rating with adequate pressure heads between nodes 10 to 20, that is, around Delta Park area. But from nodes 1 – 10, the pressure is below average line, while from nodes 20 – 24, the pressure heads are satisfactory (see Fig. 6).

OPTION 3 (CPPS running alone) developed the least pressure rating of the seven simulation runs (that is the seventh best overall rating). This is probably due to the relatively low elevation of the existing overhead tank (see Table 1).

OPTION 5 (UPPS and CPPS running simultaneously) developed adequate pressure heads elsewhere, except between nodes 10 and 19, around Delta Park area (see Fig. 6). It scored the fifth best overall rating.

OPTION 6 (UPPS and CPPS running simultaneously) was rated the sixth best, with low pressure heads between nodes 1 and 14, which are generally below the average pressure line.

OPTION 7 (UPPS, DPPS and CPPS running simultaneously) produced the fourth best pressure heads in the network. Although, some values are slightly below the average line between nodes 8 and 17.

The simulation options (1 thro 7) were ranked for best performance on the basis of frequency distribution of the highest current heads after 200 iterations. First, the current head distributions after 200 iterations for the seven options are as presented in Table –1 which served for the construction of frequency distribution. For each node in Table-1 the simulation options (1 thro 7) producing the highest current head is selected for best performance and a check-mark is indicated against it. The summation of the best performance producing the highest current heads for the 27 nodes yielded the following results: 26 out of 27 (96.3%) for option 4; 14 out of 27 (52%) for option 1, 10 out of 27 (37%) for option 2, 8 out of 27 (29.6%) for option 7 (Fig. 5).

In effect, the histogram (Fig. 5) of highest pressure heads in each node and graphical plots (Fig. 6) yielded the following pressure ranking: Option 4, Option 1, Option 2, Option 7, Option 5, Option 6 and Option 3, respectively.

3.3 Effects of Pipe Materials on Pressure Head Distribution

The equivalent sand roughness values for uPVC, wrought iron, cast iron, and asbestos-cement (Table 2) were replaced one after the other during simulation exercise [see Eq. (4)]. This is to determine what effects, if any their e-values have on the options in their earlier rankings.

The graphical plots of pressure heads against selected nodes for each of the materials listed in Table 2 is shown in Figs. 7a – 7d, respectively. Option 4 still maintains the highest pressure heads, followed by Option 1 and lastly Option 7. This shows that the pipe material used in water distribution does not affect the options in their earlier ranking, and thus the pressure regime in the network. Ordinarily, pipe material may influence pressure head distribution but not significantly as to alter the ranking achieved via overhead tank elevation(s).

As per option 4 (the best option), the test runs were repeated in order to establish which of these materials: uPVC, wrought iron, cast iron and asbestos-cement will give the maximum pressure head development in the network. This was done by substituting the equivalent sand roughness (e) value for each of the selected materials above into Equation (4) in the computer program. The plots of computed pressure heads against the nodes for each material are as shown in Fig. 8. The results re-affirms that pressure development in water distribution network depends on friction factor of the pipe material, which in turn depend on surface roughness, relative roughness and other parameters.

3.4 Effect of Step-Wise Increase in Overhead Tank Elevations on Pressure Head Distribution

Using the best option (4), the elevations of the three reservoirs: Unipark, Delta Park and Choba Park were increased in steps of one metre. The pressure head responses for each step-wise increment was plotted against selected nodes (Fig. 9). From the plots, there is proportional increase in pressure head for every one metre increase in reservoir elevation in accordance with the principles of hydrostatic pressure head variation. Nevertheless, the pressure response has no impact at node 10 (dead end at University gate near AP filling station). This is probably due to friction dampening of the long pipe length. At node 19 (the location of DPPS), there is pronounced pressure head response which decayed gradually to node 21 (entrance to CPPS). Beyond this node, the effect of step-wise increase in reservoir elevation was not felt.

4. CONCLUSION

The water distribution network modeling is merely the simulation of pressure heads and discharges throughout the network for various combinations of OHT elevations. The analysis of UNIPORT water distribution system has been carried out by computer simulation, in which the WASDIM program was employed as a predictive model. Based on the results of this study, the following conclusions can be drawn:

1) It is possible for the three Uniport campuses to have an integrated water supply distribution network for sake of economy, that is, savings in running cost of three independent pumping stations as existing prior to this study.

2) The best three options are: Option 4: Unipark and Delta Park pumping stations running simultaneously; Option 1: Unipark pumping station running alone; and Option 2: Delta Park pumping stations running alone.

3) In terms of pressure head and discharge ratings, any of the materials; uPVC, wrought iron, cast iron and asbestos cement may be used, as the pressure head variations are insignificant.

4) The final choice of any of the pipe materials depends on its availability, cost, funds and environmental factors.

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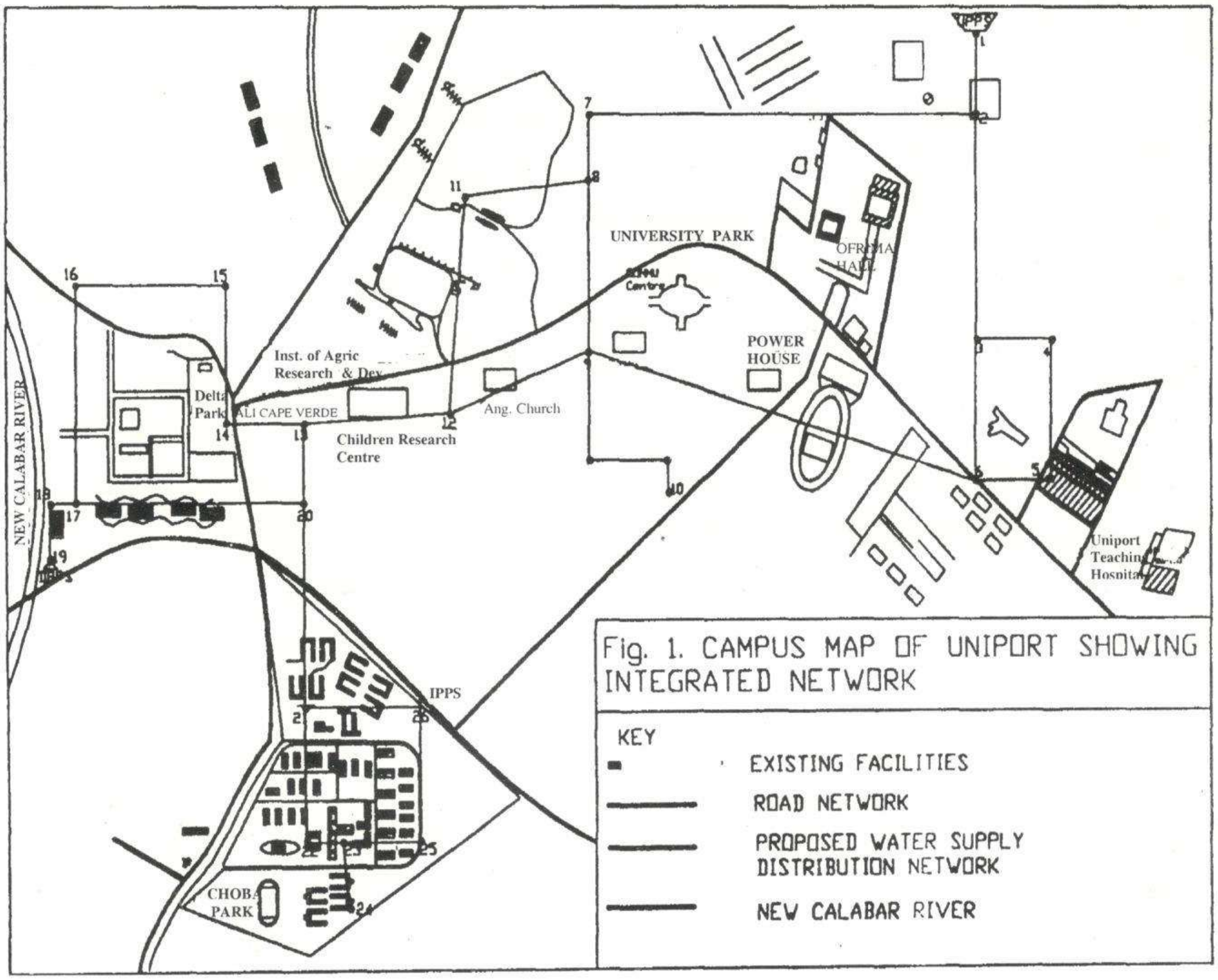


Fig. 1. CAMPUS MAP OF UNIPORT SHOWING INTEGRATED NETWORK

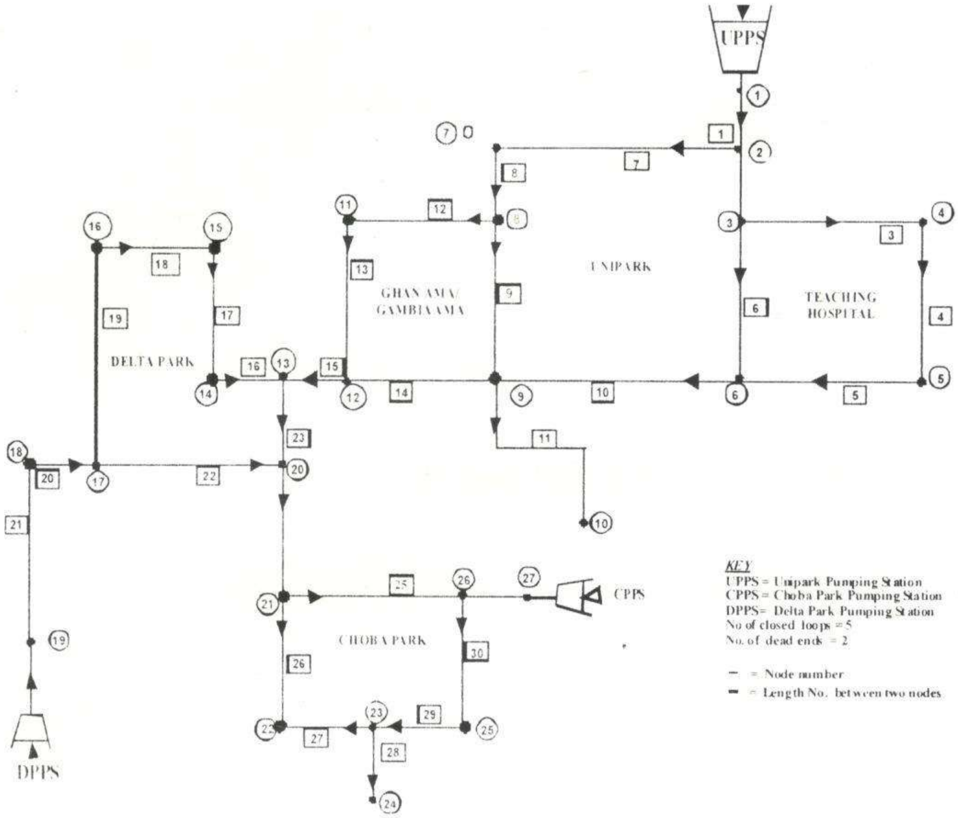
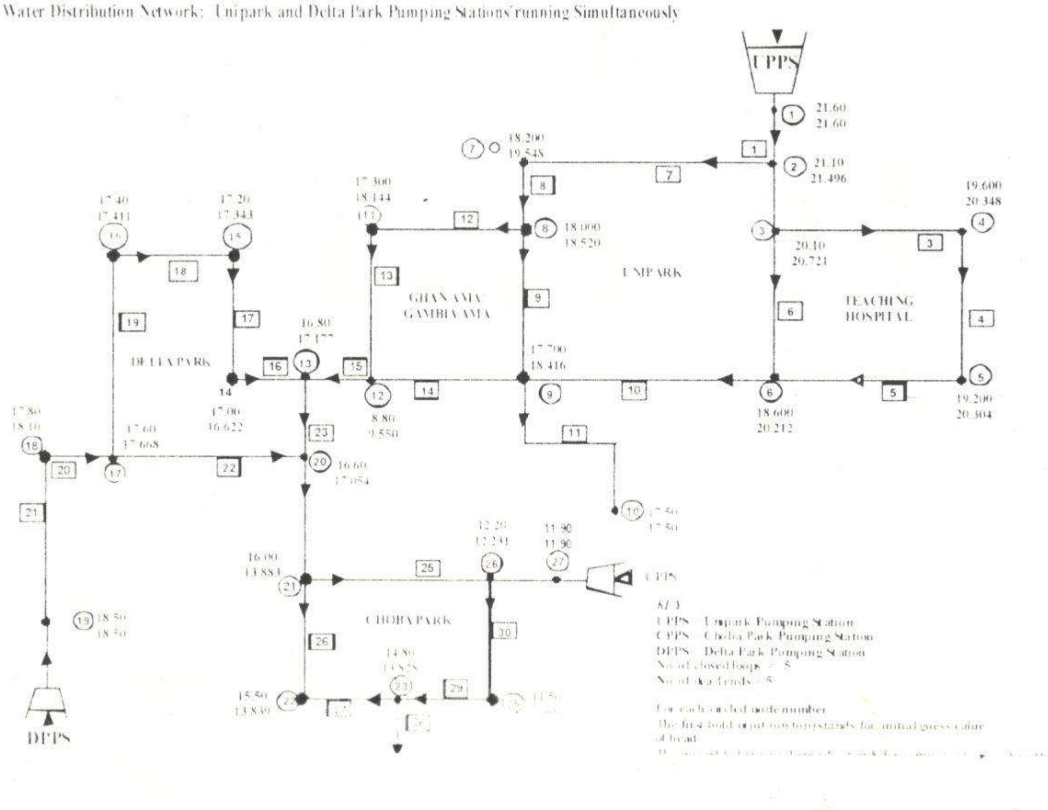


Fig. 2 : Water Distribution Network: Unipark and Delta Park Pumping Stations'running Simultaneously



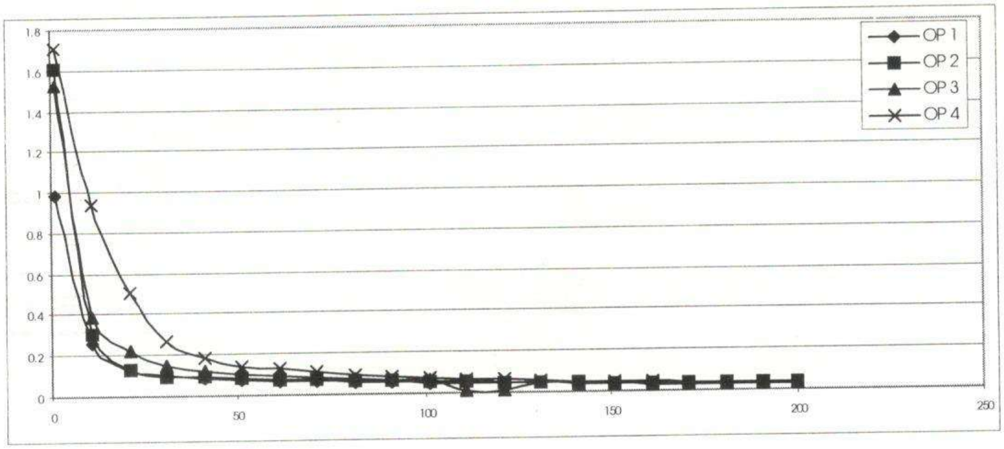


Fig. 4: The first 4 Options (Computed error on Delta – H)

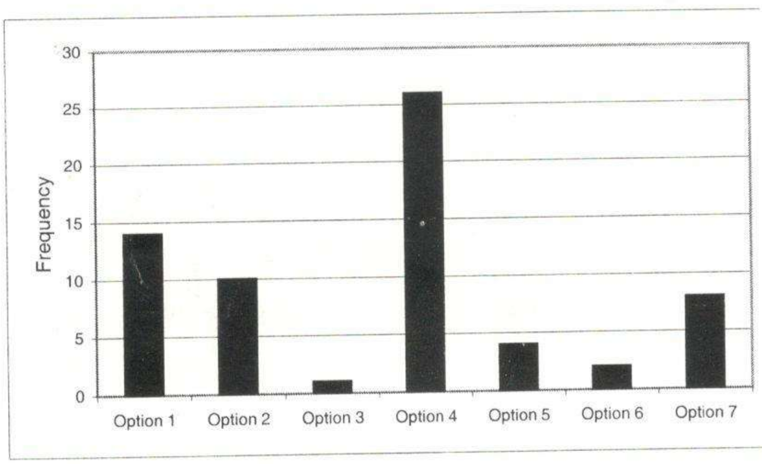


Fig.5: Frequency Distribution of Highest Pressure Heads After 200 iteration

Table 1: Current Heads After 200 Iterations

nodes	op1±	op2	op3	op4	op5	op6	op7
1	21.60	9.00	6.60	21.60	21.6	7.00	21.60
2	21.47	9.21	6.65	21.50	21.46	7.08	21.47
3	20.45	10.64	7.00	20.72	20.55	7.74	20.64
4	20.05	11.26	7.13	20.35	20.20	8.09	20.37
5	19.99	11.35	7.15	20.30	20.13	8.13	20.31
6	19.84	11.66	7.26	20.21	19.86	8.14	19.98
7	18.93	13.31	7.78	19.55	18.52	8.42	18.47
8	17.56	15.49	8.39	18.52	16.92	9.15	16.86
9	17.38	15.78	8.43	18.42	16.80	9.18	16.82
10	15.20	16.20	8.40	17.50	16.60	8.00	19.30
11	17.05	16.18	8.74	18.14	16.091	9.73	15.54
12	15.87	16.20	9.55	17.35	14.18	11.09	12.61
13	15.58	16.52	9.76	17.18	13.71	11.23	11.94
14	15.58	16.95	9.75	17.20	13.71	12.09	11.58
15	13.31	16.95	9.79	17.34	13.00	14.59	11.58
16	15.15	17.10	9.82	17.41	12.66	15.19	12.16
17	14.44	17.41	9.99	17.67	11.40	15.98	14.22
18	13.70	17.98	10.01	18.10	9.95	17.37	16.54
19	13.00	18.50	10.20	18.50	8.60	18.50	18.50
20	15.05	16.50	9.97	17.05	12.89	14.12	12.05
21	13.87	13.84	11.30	13.88	11.80	11.58	11.60
22	13.87	13.82	11.36	13.84	11.17	10.91	11.10
23	13.85	13.81	11.38	13.83	11.17	10.88	11.10
24	13.90	13.90	11.30	13.90	11.30	10.00	10.00
25	12.78	12.76	11.65	12.77	11.62	11.46	11.56
26	12.23	12.23	11.80	12.23	11.85	11.78	11.81
27	11.90	11.90	11.90	11.90	11.90	11.90	11.90

±OP1 = OPTION 1

Table 2: Equivalent sand roughness (e) for selected pipe materials

S/No.	Material	Mm
1	uPVC	0.03
2	Wrought Iron	0.06
3	Cast Iron	0.30
4	Asbestos-Cement	0.02

Fig. 6: Plot of Pressures Vs Selected nodes

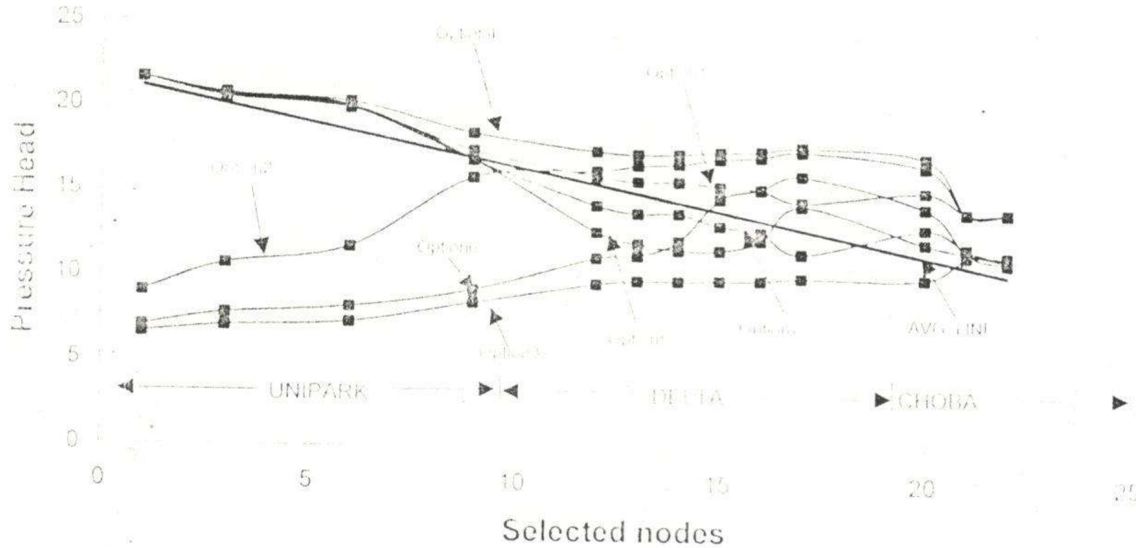


Fig. 7a: PLOT OF PRESSURE HEADS VS SELECTED NODES USING uPVC

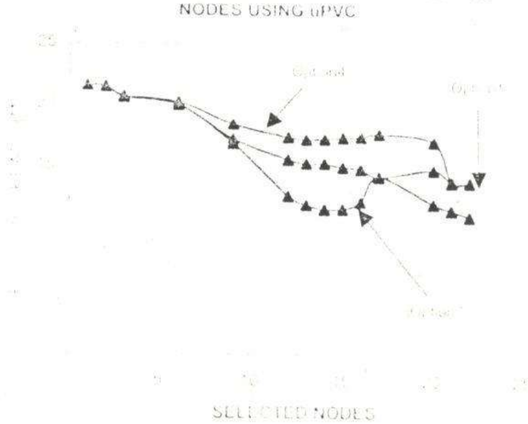


Fig. 7b: PLOT OF PRESSURE HEADS VS SELECTED NODES USING WROUGHT IRON

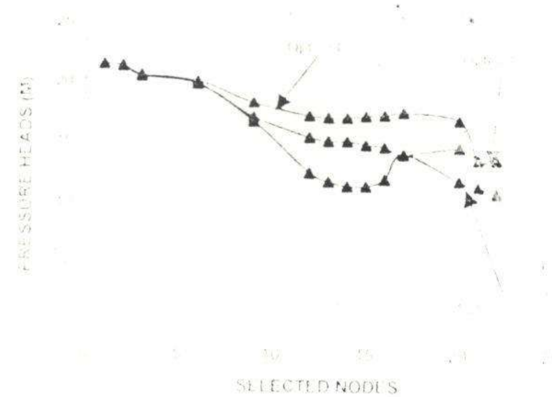


Fig. 7c: PLOT OF PRESSURE HEADS VS SELECTED NODES FOR CAST IRON

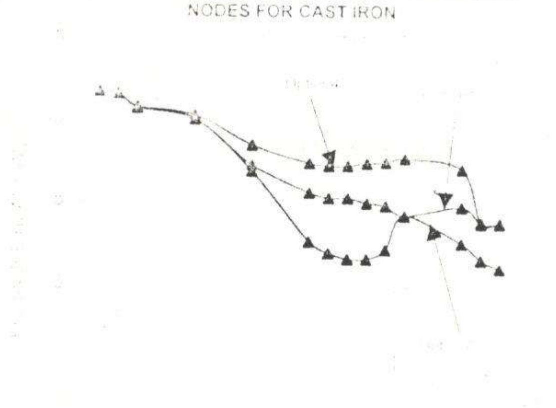
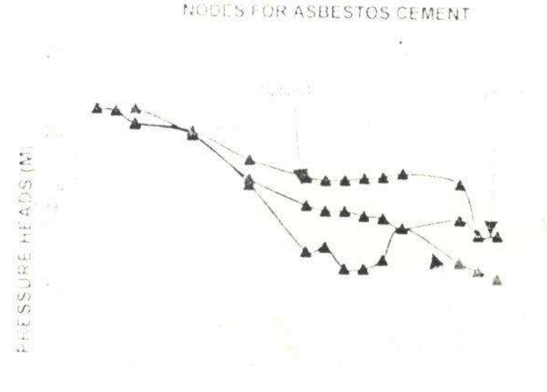


Fig. 7d: PLOT OF PRESSURE HEADS VS SELECTED NODES FOR ASBESTOS CEMENT



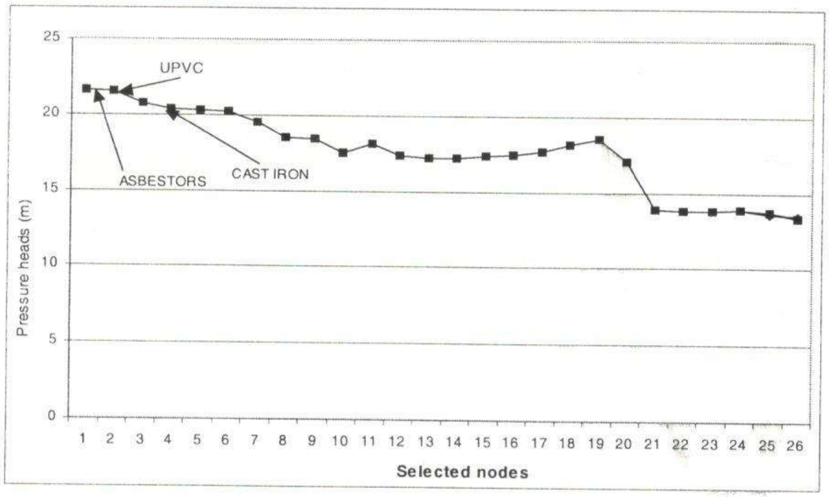


Fig. 8: Pressure Heads Distribution for Option 4

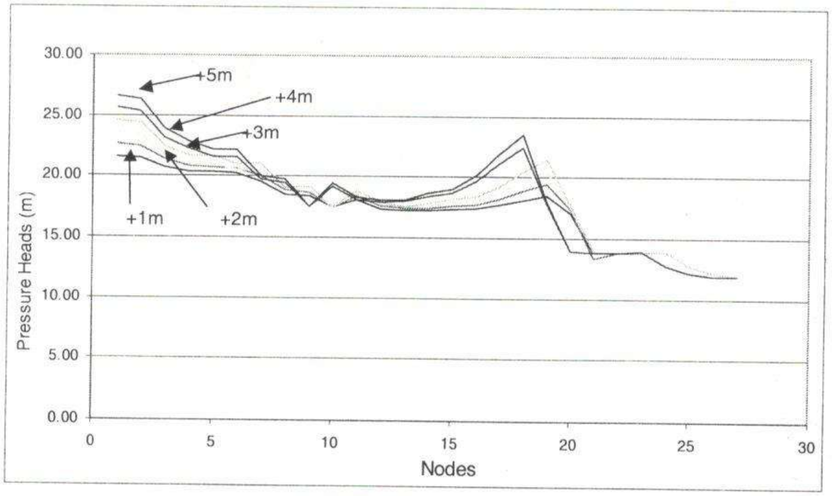


Fig. 9: Pressure Head Response of Option 4