APPLICATION OF LINEAR THEORY METHOD ON AHMADU BELLO UNIVERSITY WATER DISTRIBUTION NETWORK

M.Sc. THESIS

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CERTIFICATION

This project thesis entitled "Application of Linear Theory Method on Ahmadu Bello University Water Distribution Network" by Adembawa Sulaima Lebbe, meets the regulations governing the award of the degree of M.Sc. (Water Resources and Environmental Engineering) of Ahmadu Bello University and is approved for its contribution to knowledge and literary presentation.

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DEDICATION

This thesis is dedicated to my wife

Mrs. Nazeera Sulaima Lebbe

for her forebearance, impressionable loyalty, incredible capacity for hard work and a good heart.

DECLARATION

I hereby declare that this thesis has been written by myself and that it is a record of my own research work at Ahmadu Bello University, Zaria. It has not been accepted in any previous application for a higher degree.

All sources of information have been specifically acknowledged in the reference section.

A. SULAIMA LEBBE

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ABSTRACT

In this thesis it is presented that the application of Linear theory method in solving hydraulic problems of distribution networks to the Ahmadu Bello University Water Supply Pipe Network with the view to improve the distribution system in the high pressure zone.

Computer analysis in Fortranlanguage were carried out for pipe networks of all residential areas to obtain the head losses and flowrates in pipes to determine the adequacy of the pipe network. Pressure head distributions and operational pressure head required at feeding points of pipe networks were computed to determine the suitability of the height of existing tanks

Computations were carried out to determine the diameter of supply lines which will give the required distribution of water from the central over head tank by gravity without any further control. The required capacity of the central over head tank was also estimated.

Finally, influences of minor losses, changes in roughness of pipe, changes in consumption rate, on distribution of pressure heads and the degree of simplification of pipe network were determined.

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LIST OF SYMBOLS

a	=	exponent for head loss relationship;
D	=	pipe diameter; (m)
E	=	roughness; (cm)
f	=	friction factor;
g	=	acceleration due to gravity;(m/sec^2)
h_L	=	head loss in pipe; (m)
h_{Li}	=	head loss in pipe i;(m)
h_{min}	=	Standard minimum pressure head;(m)
H_e^n	=	hydraulic gradient line elevation at node 'n' when pressure head at feeding point is zero;(m)
H_e^{up}	=	hydraulic gradient line elevation at the upstream node when the pressure head at feeding point is zero; (m)
H_{opr}^n	=	operational pressure head at node 'n';(m)
H_r	=	relative pressure head at node 'n';(m)
J	=	number of junction in network;
K_i	=	loss coefficient in pipe i; (m^{-2} s^{-1})
K_i'	=	modified loss coefficient in pipe 'n'; (m^{-2} s^{-1})
l	=	number of loops in network;
L	=	pipe length; (m)
Δ_L	=	minor loss in pipe;(m)
m	=	node number at feeding point;
n	=	node number;
N	=	number of pipes in network;
q	=	consumption rate per house (l/s)

Q_{in} = flow into network; (l/s)

Q_{out} = flow out of network;(l/s)

Q_i = flow in pipe i; (l/s)

Q_{i-1} = flow in pipe i for previous trial; (l/s)

Q_{i-2} = flow in pipe i for trial before previous one; (l/s)

Q_{io} = discharge in pipe i used to compute modified pipe line constant; (l/s)

Re = Reynold number

v = velocity of flow in pipe (m/s)

Z^n = topo level at node 'n'; (m)

ν = coefficient of kincmatic viscosity; (m^2/s)

CHAPTER 1

INTRODUCTION

1.1. Purpose of the study:

The purpose of the study is to investigate the application of linear theory method for hydraulic computations of pipe network to the Ahmadu Bello University water Supply with the view to improve the distribution system in nigh pressure zone in the residential areas of Samaru Campus.

1.2 Description of existing system:

Residential areas of Samaru Campus has two sources of supply. These are Ahmadu Bello University (A.B.U.) Water Works and Kaduna State Water Board (K.S.W.B.).

A.B.U. Water Works currently supplies water to all residential areas except residential areas area 'C', Site II and Silver Jublee Estate.

The source of raw water to A.B.U. Water Works is from a reservoir formed by constructing a earth dam on river Kubanni. According to the study plan prepared by Melyepterv Consultants (1972) the expected water demand as at year 2000 was 9000m^3 per day. Tnis demand was design to fulfill in two stages of constructions.

Tne first stage is to produce 4500m^3 per day while the second stage is to double its capacity to 9000m^3 per day.

The raw water from the reservoir is drawn via an intake pump house where water is pumped to the water works through two parallel lines of 350mm steel pipes. One line is sufficient to meet the first stage while the second line was provided in advance to meet the second phase demand of the project.

As a first stage in the process of purifying technology, the raw water is exposed to intensive aeration to saturate the raw water with oxygen and ventilate certain amount of volatile materials to eleminate the odour and by - flavour. The designed duration of this treatment is approximately 10 minutes, provides sufficient reaction - time for the dissolved oxygen to oxidise the non-volatile materials or to render them more volatile.

In the second stage, the aerated raw water is treated with chlorine, Alluminium sulphate, and Hydrated Lime. According to the design, the maximum chlorine dose is $5g/m^3$ while $80g/m^3$ for alluminium sulphate and $20g/m^3$ for hydrated lime.

Then in the third stage of treatment process, the water treated with chemicals is allowed to settling tanks where the water flows upwards from the bottom while leaving its floatings and coagulated colloid contaminants. The settling tank has a total volume of $400m^3$ with retention time of 2 hours.

In the fourth stage of the treatment process, the settled water is allowed from the clarifier to quick sand filters where the filters remove the residual floating materials from the settled water.

This system has a facility to do back washing with water and air when the filters are clogged by the contaminating materials.

Finally, a provision has been made to dose chemicals to filtered water for further correction on residual chlorine and PH value before it passes to the clear water wells. The total capacity of the clear water well is 1500m^3.

The present supply level of distribution was achieved through a number of stages. Originally, the Main Campus and area 'A' (excluding the area near the Police gate) were isolated for A.B.U. supply. Later within a few months A.B.U. supply was extended to area BZ and Police gate area.

As from October 1983, areas F, E and G were connected to the A.B.U. Supply through a Booster Station which is located between Faculty of Medicine and area BZ. Supply to these areas are limitted due to inadequate pumping main and the level of production of water at treatment works.

At present, residential areas BZ, E, F, G and A.B.R.L.S. Houses are supplied with water in rationing turns, while area 'A' gets almost round the clock daily.

The supply situation in area 'C' is unpredictable and well below satisfaction due to deteriorating conditions of Kaduna State Water Board. As at now there is no independent pumping main to area 'C' from A.B.U. water works except the one through area 'G' which was connect d with the view to ease the situation in area 'C' by supplying water from area 'G'.

Area 'F' has its own 225m^3 capacity steel over head tank while areas G and E have a common over head tank of 225m^3 capacity. The tank at area 'F' is satisfactorily used while the common tank at E and G is used as an additional storage for those areas.

From the records at A.B.U. Water Works, there was a satisfactory water supply in the main campus, area 'A' and area BZ round the clock daily before all other residential areas were connected to A.B.U. Supply through its Booster Station. - This is so, even now when over the booster is faulty and subsequently water is supplied only to those areas. Generally, all residential areas in Samaru Campus supplied from A.B.U. water works could be catagorised in two main groups namely; low pressure zone and high pressure zone, as shown in appendix 'A'.

1.2.1 Low pressure zone:

Main Campus, residential areas 'A' and BZ are considered as in low pressure zone where the existing over head tank with the capacity of 270m^3 near Senate Building is feeding the entire areas by gravity, assumed to have no difficulties in achieving a satisfactory level of supply round the clock as it was the case before the function of A.B.U. Booster Station.

1.2.2 High pressure zone:

Residential areas C, E, F, G and A.E.R.L.S. Houses were considered as areas in high pressure zone. These areas are fed with water on its rationing time through A.B.U. Booster Station, except area 'C' for which water is supplied by K.S.W.B.

Guide maps of areas C,E,F & G are given in Appendix 'B'.

1.3 Identification of technical problems:

As at now areas under rationing from A.B.U. Water Works are B2, E, F and G. All these areas including area 'C' supplied by K.S.W.B. are considered to be with no supply or supply with inadequate quantity and pressure or inadequate duration of supply.

Existing over nead tanks in these residential areas do not have seperate direct supply lines or pumping mains from the A.B.U. Booster Station or the source of supply. The only main existing is the one supplying to all these areas at different periods. It is in this regard that the inadequate pumping mains have been identified as one of the causes of insufficient water supply to these areas.

The second problem was inadequate pressures at the consumers end from their over nead tanks, particularly from house Nos. 1 to 12 in area F, A.E.R.L.S. houses and all block of flats in area 'G'.

The third problem was contributed by frequent closing, opening and regulating the main valves in the pumping main.

1.4 Formulation of research problem:

Out of the problems identified above, the distribution system of residential areas C, E, F, G and A.E.R.L.S. houses were taken for analysis with a basic assumption of a system with a central over head tank with a view to improve the supply by gravity round the clock daily from the proposed central over head tank. The problem is formulated as follows;

- Hydraulic computation of pipe networks of different areas
- Location of supply points for different areas
- Design of the supply lines from the central over head tank to different pipe networks
- Evalution of the height of the central over head tank
- Determination of the influencies of minor losses on distribution of pressure heads
- Determination of the influences of changes in roughness of the pipe on distribution of pressure heads
- Determination of the influence of the changes in consumption rate on distribution of pressure heads
- Determination of the degree of symplification of networks.

CHAPTER 2

LITERATURE REVIEW AND CHOICE OF METHOD

In closed networks, the analysis of hydraulic problems has received considerable attention in the past. This is almost entirely due to the available computer facilities which makes tedious, iterative calculations more amenable to a quick solution. In addition, obtaining a solution to these problems has important economic and design significance.

One of the first and probably the most widely used method of analysis is the Hardy Cross Technique (1936). This method makes corrections to initial assumed values by using a first order expansion of the energy equation in terms of correction factor for the flowrate in each loop. The process is, of course repetitive and is dependent on the accuracy of the initial guess which must be reasonably good if an answer is to be obtained rapidly. However, the method is suitable for hand calculation and also a number of digital programs have been prepared for network flow analysis using this procedure. In certain cases it has been found that the Hardy Cross method converges very slowly or not at all. This led McCormick and Bellamy (1968) to suggest special measures to improve convergence.

A second method which is being applied successfully to hydraulic network analysis utilizes Newton-Raphson method to formulate a set of simultaneous linear equations which can be solved for flow corrections for each loop in the network.

This method described by Martin and Peters (1963) has also been used by Giudice (1965) and Pitchai (1966) for studies for hydraulic networks. The method has been extended by Shamir and Howard (1968) to include various hydraulic components in the network. Epp and Fowler (1970) later reported an approach to solving hydraulic network problems utilizing the Newton-Raphson method and have offered some details pertaining to a general digital computer program available for this problem. Because this method adjusts the flowrate in all the loops simultaneously, convergence using Newton-Raphson approach is much quicker than that obtained using Hardy-Cross analysis. This is specially important when analysing networks involving large number of pipes. However, both methods of analysis requires initial guesses for flow distribution and very bad estimates of these values can lead to slow convergence or, in some cases a situation where the successive trials do not converge and the solution cannot be found.

Other methods have been proposed for hydraulic network problems but have not gained wide acceptance. Direct electrical analogs are also used for hydraulic network analysis. Most popular is the fluid network analyzer developed by McIllroy (1950). This and other available direct analog devices are described in a paper by Mcpherson (1962). Although, electrical analogs are accurates and no problems of convergence exist, special equipment and

trained operators are required to use these device which makes these methods of analysis impractical in many situations.

Wood D.J. and Chales C.O. (1972) proposed, to solve hydraulic network problems using Linear network theory which is modified to account for nonlinear effects inherent in hydraulic system. There are many advantages to this approach which make it attractive, probably the most significant is that convergence to the final result is very rapid and seems assured in all cases. The method does not require initial estimates of flowrates and is directly applicable for closed type of networks. In addition, this approach is well suited for computer programming and short, relatively simple program can be written for the method.

Later Jeppson R.W. (1976) presented the use of Linear theory method for solving the system of equations flowrates are unknown using relatively short computer program in Fortran language. The facility of this program which make it more attractive to use the Linear theory method.

Therefore, it is proposed herein to solve hydraulic network problems of A.B.U. Water Supply using Linear Theory method in computer program as in subsequent chapters 3 and 4.

CHAPTER 3

HYDRAULIC COMPUTATION OF NETWORKS

3.1 Basic relationships:

Pipe network problem is solved by obtaining a flow distribution which satisfies flow continuity at junctions and conservation of energy arround any closed loop. The continuity relationships are linear algebraic equations while the conservation of energy equations are generally nonlinear and no method for direct simulteneous solution of these equations is known.

Most solutions begins by assuming values for flows which satisfy continuity conditions. Then corrections are made to these assumptions utilizing the energy equations in accordance with some predetermined method until a solution for flowrates within a required tolearance is reached in which both continuity and conservation of energy conditions are satisfied.

Let the network has N pipes, J junctions and ℓ loops, then in all junctions in the network the discharge into the junction equals the discharge out of the junction:

$$Q_{in} = Q_{out} \quad .. \quad .. \quad .. \quad .. \quad (3.1)$$

in which Q = Volumatic Discharge. It is possible to write J linear coninuity equations out of which J - 1 are independent and the balance is redundant. Similarly ℓ nonlinear energy equations could be written, one for each loop.

$$\sum_{i=1}^{\lambda} h_L = 0 \quad .. \quad .. \quad .. \quad .. \quad (3.2)$$

in which h_L = head loss in a pipe contained in that loop, and is a function of discharge, Q.

Therefore from equations (3.1) and (3.2), N simulteneous equations are obtained in terms of discharge in each pipe. Head loss in a pipe i is best represented by a relationship:

$$h_{Li} = K_i Q_i^a \quad .. \quad .. \quad .. \quad .. \quad (3.3)$$

in which K_i = a Constant normally a function of pipe length, diameter, and type of pipe materials, could be calculated from Darcy-Weisbach equation which is normally written as follow:

$$h_L = f \frac{L}{D} \frac{V^2}{2g}$$

$$\therefore \quad h_L = f \frac{8L}{g\pi^2 D^5} Q^2 \quad .. \quad .. \quad .. \quad (3.4)$$

where V = velocity of flow in pipe

f = friction factor of the pipe

From equations 3.3 & 3.4 for a given pipe i

$$K_i = f_i \frac{8L}{g\pi^2 D_i^5} \quad .. \quad .. \quad .. \quad (3.5)$$

and a = an empirical head loss exponent usually 2.0

This relationship makes each of the ℓ loop equations nonlinear and no method is known for the direct solution of these set of simulteneous equations.

In network with closed loops, open-tree type or combination of both types the following identity holds:

$$N = J + \ell - 1 \quad .. \quad .. \quad .. \quad .. \quad (3.6)$$

3.2 Linear theory method (Ref. Wood D.J. and Chales C.O. (1972)

In this method the nonlinear loop equations are transformed into linear equations by approximating the

head loss by

$$h_{Li} = K_i \; Q^a = K_i \; Q_{io}^{a-1} \; Q_i = K_i' \; Q_i \quad \ldots \quad (3.7)$$

in which Q_{io} = the approximate discharge in line i.

Using the values for approximate discharges to compute the modified constant, K_i, the loop equations can be expreseed as linear equations which when combined with continuity equations yield N linear simultaneous equations which can be readily solved for discharge in each line.

However, this is an approx. solution as approx. values for discharge are used to linearize the head loss terms. The computed values for discharge can then be used to compute new values for modified constant; K_i which are used to obtain a new set of or simultaneous equations which can be solved for improved values for pipe discharges. This process can be continued until the discharge obtain from two successive sets of calculations do not differ significantly.

3.3 Features of the method:

Several features of this method which make it more attractive over all other methods for hydraulic network computations.

3.3.1 Calculation of initial flowrates:

In all other methods presently utilized for hydraulic computation require an initial estimate of flowrate in each pipe. Poor estimation can greatly affect the convergency of the solution and some time leads to a situation where solution will not converge. In this method it is not necessary to estimate an initial flowrates instead, an accurate

initial flowrate can be easily calculated. This is done by assuming that the modified constant is independent of flowrate and as a first approximation is given by

$$K_i' = K_i \quad .. \quad .. \quad .. \quad .. \quad (3.8)$$

using Equation 3.8 a linear set of equations can be obtained and solved simulteneously for discharge.

3.3.2 Convergence of solution:

In this method, he observed that successive trials always gave a result which converged to the correct solution, but the results of successive trials tended to oscillate about the final result. It is also said that the average of two successive trials gave a result very close to the final value of flowrate.

Therefore, the average value of the two prior sets of calculations for flow rate were used to compute the best value of the discharge for that trial and modified constent K_i' which is employed for the next trial. This is expressed as

$$Q_{io} = \frac{Q_{i-1} + Q_{i-2}}{2} \quad .. \quad .. \quad .. \quad (3.9)$$

where Q_{i-1} = flowrate obtained from previous trial for pipe i.

Q_{i-2} = flowrate obtained for the trial previous to that.

This averaging process produces a very rapid convergence to the solution and it is unlikely that more than a few iterations.

3.3.3 Computer programming method:

Suitability of the method for computer programming is an important factor. Unlike any other method ,

this technique do not requires identification of pipes common to two loops. Common pipes are of no special significance to the formulation of junction and loop equations.

Utilizing available sub-routines for the solution of simulteneous linear equation it is possible to write a very short Fortran Computer program which will handle any type of hydraulic network with the small amount of input data.

3.4 Mathmatical formulation of problem:

All hydraulic networks are described in essentially the same manner. As it was described before a network of N pipes, J junctions and ℓ loops will be described by J-1 junction equations and ℓ loop equations. If each pipe is assigned to have a code number, the geometry of the network could be fully described by (J-1) sets of data giving the code numbers of the connecting pipes at all but not one of the junction and ℓ sets of data giving the code numbers of the pipes arround each closed loop.

Further information can be prescribed if a sign is used with code number. For junction equations, a positive sign for the code number means that the assumed flow direction is out of the junction in that pipe and a negative sign on the code number means the assumed flow direction is into the junction. For loops, a positive sign on the code number means flow is in the clock wise sence arround that loop.

In addition a term representing flow into or out of the network at each junction can be added to the code

numbers of the pipes representing that junction to complete all the informations, exclusive of pipe line data, needed to describe the network.

Junction numbers and loop numbers do not have to be provided as data. After each pipe in a network has been assigned a code number and a direction, it is simple to prepare the necessary data describing the network.

Only data needed in addition to above is a statement for each pipe giving length, diameter, and roughness.

Now the network of N pipe is completly described by 2N sets of data. The main goal of the programming is to define the coefficient of an N x N matrix (a) and an N dimensional vector $\bar{Q}$ which represent the linear simulteneous network equations expressed as

$$\bar{a}\ (i, k) \times \bar{Q}\ (k) = \bar{B}\ (i) \quad .. \quad .. \quad (3.10)$$

where $i = 1$ to N

$k = 1$ to N and

$N = J + \ell - 1$

For the J-1 junction equations, the matrix coeffient $a(i, k) = 0$ for a junction if k is not identical to one of the code numbers for the pipes connecting at that junction.

If k is equal to a code number, $a\ (i, k) = 1.0$

If code number is negative, $a\ (i, k) = -1.0$.

The value of the vector B(i) is equal to external network discharge at that junction.

The J-1 continuity equations generated in this manner do not change from trial to trial.

For loop equations a(i, k) must be defined for the region i = J to i = N and can be computed directly from the loop data.

For any loop a(i, k) = 0 if k is not identical to one of the code numbers of the pipes composing that loop.

If k is identical to a code number, a(i, k) = K'(k)

If code number is negative, a(i, k) = - K'(k).

The elements of vector B are zero for all the loop equations.

For successive trials coefficients K' of the loop equations changes as they are dependent on the flowrate.

3.5 Data for the original program:

The original program was taken from Jeppson (1981) pp 75 -78.

Basically four different types of informations are required. First, number of pipes NP, number of junctions NJ and number of loops l, in the network plus other specifications such as denoting type of data (i.e. ES or SI) number of allowable iterations, viscosity of fluids and the maximum error in flowrate that could be permitted in two successive iterations.

Second, data giving diameters, lengths and wall roughness for each pipe .

Third, information is for establishing junction continuity equations. This information is provided by a set of data for each junction which contains the pipe numbers meeting at the junction with its relevant sign.

Fourth, information is required for energy equations. For each loop in the network, this information lists the pipe numbers in that loop with its relevant sign.

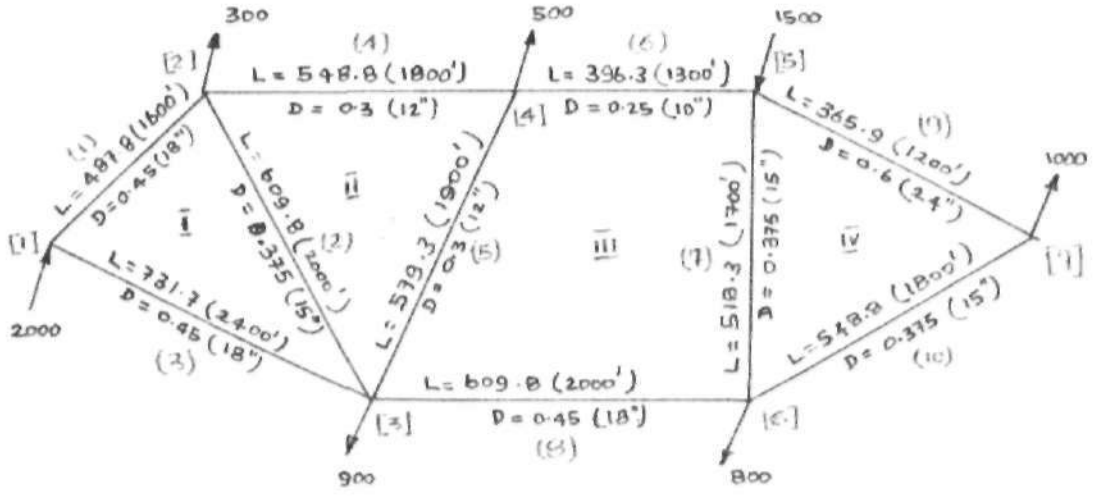
3.6 Results from the program:

Generally this program has facilities to give the results of the following parameters in the given order.

1. Pipe diameters
2. Length of pipes
3. Relative roughness of pipes
4. Number of iterations
5. Flowrate in pipes
6. Head losses in pipes
7. Flowrate in pipes (only for English unit in GPM)
8. Warning - over flow occured (due to redundant equation resulting in singular matrix).

3.7 Demonstration with an example:

In order to illustrate the analysis of flow distribution in a hydraulic network using the program a known relatively simple 10 pipesflow distribution network depicted in Fig. 3.1. This schematic shows the discharge into and out of the network, along with assumed flow direction in the pipes, node numbers, loop numbers and pipe parameters like lengths and diameters. Roughness of the pipes are given below in the same schematic diagram.



e = 0.0102" Except

e_6 = 0.009" and

e_8 = 0.009" Water at $68^{\circ}F$

: L = Length (m)

: D = Diameter (m)

: f = Flow at node (gpm)

[ ] : Node No.

( ) : Pipe No.

I , II , III : Loop No.

FIG. 3.1 NETWORK DIAGRAM OF THE EXAMPLE

3.7.1 Equations and matrices:

The first set of equations satisfying the continuity at junctions are as follows:

$$
\begin{aligned}
Q_1 + Q_3 &= 4.46 \\
Q_1 - Q_2 - Q_4 &= 0.67 \\
Q_2 + Q_3 + Q_5 - Q_8 &= 2.01 \\
Q_4 - Q_5 + Q_6 &= 1.11 \\
Q_6 + Q_7 + Q_9 &= 3.34 \\
Q_7 + Q_8 + Q_{10} &= 1.78 \\
Q_9 - Q_{10} &= 2.23
\end{aligned}
\qquad (3.11)
$$

where Q is in $ft^3/sec.$

The second set of equations are the energy equations in loops. They are as follows:

$$
\begin{aligned}
K_1 Q_1 + K_2 Q_2 - K_3 Q_3 &= 0 \\
K_4 Q_4 + K_5 Q_5 - K_2 Q_2 &= 0 \\
-K_6 Q_6 + K_7 Q_7 - K_8 Q_8 - K_5 Q_5 &= 0 \\
K_9 Q_9 + K_{10} Q_{10} - K_7 Q_7 &= 0
\end{aligned}
\qquad (3.12)
$$

Now the corresponding matrixes are as follows:

$$\begin{bmatrix} 1 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 1 & -1 & 0 & -1 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 1 & 0 & 1 & 0 & 0 & -1 & 0 & 0 \\ 0 & 0 & 0 & 1 & -1 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 & 1 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 & 1 & 0 & 1 \\ K_1 & K_2 & -K_3 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -K_2 & 0 & K_4 & K_5 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & -K_5 & -K_6 & K_7 & -K_8 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & -K_7 & 0 & K_9 & K_{10} \end{bmatrix} \begin{bmatrix} Q_1 \\ Q_2 \\ Q_3 \\ Q_4 \\ Q_5 \\ Q_6 \\ Q_7 \\ Q_8 \\ Q_9 \\ Q_{10} \end{bmatrix} = \begin{bmatrix} 4.46 \\ 0.67 \\ 2.01 \\ 1.11 \\ 3.34 \\ 1.78 \\ 0 \\ 0 \\ 0 \\ 0 \end{bmatrix}$$

On the process of computation it will be reduces to 10 x 10 matrix as six out of seven continuity equations are independent. (3.13)

3.7.2 Read statements in the original program:

It has altogether 7 Read Statements. They are as follows:

First, NP, NJ, NL, max, unit, error, ΔQ

where NP = Number of pipes

NJ = Number of junction

NL = Number of loops

max = maximum iteration allowed

unit= unit of measurement:-

Unit = 0, if D & E in inches

Unit = 1, if D & E in feets

Unit = 2, if D & E in meters

error = The maximum differences that would be allowable in two subsequent trials

Q = Difference of flowrate between the first and the second trials.

Second, Pipe Diameters (D) for i = 1 to NP

Third, Pipe Lengths (L) for i = 1 to NP

Fourth, Pipe roughnesses (E) for i = 1 to NP

Fifth, Iflow, NNJ, JN (from 1 to NNJ)

where If, Iflow =0 - no inflow

If Iflow =1 - the next card gives magnitude in GPM

if Iflow =2 - the next card gives magnitude in CFS

If Iflow =3 - the next card gives magnitude in CMS

NNJ = No. of pipes at junctions - positive for inflow and negative for out flow

JN = The number of pipes at junction If flow enters minimum and If flow leaves the pipe number is positive.

Sixth, QJ if Iflow is greater than 0

Seventh, NNJ, LP (from 1 to NNJ)

3.7.3 Input data for the example:

The data for the example is as shown in Fig. 3.2 and the input data prepared for the above example, as follows:

First, 10 07 04 10 00 0.001 0.00001217 0.1

Second, 18. 15. 18. 12. 12. 10. 15. 18. 24. 15.

Third, 1600. 2000. 2400. 1800. 1900. 1300. 1700. 2000. 1200. 1800.

Fourth, .0102 .0102 .0102 .0102 .0102 .009 .0102 .0102

Fifth; 01 02 01 03

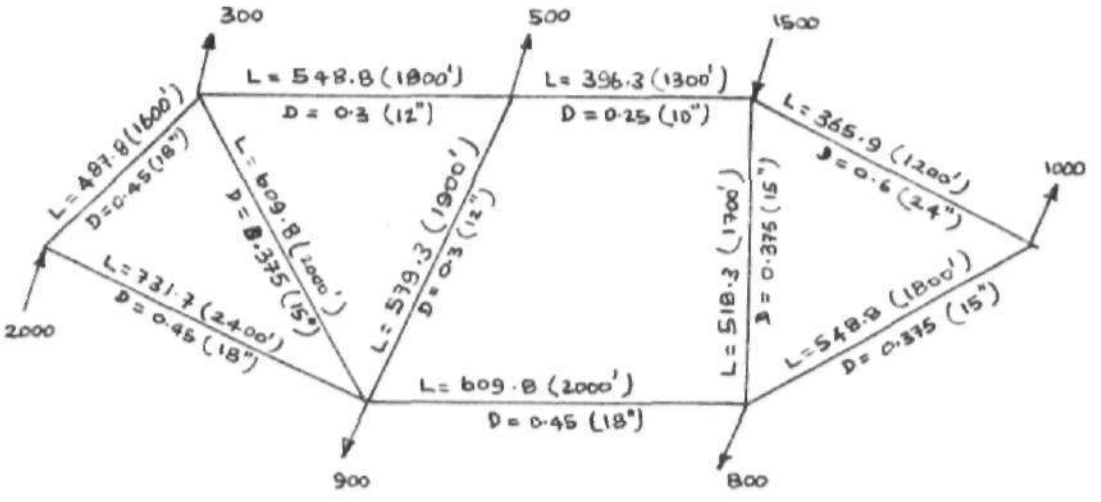
and

Sixth, 2000.

01 03 -1 02 04

-300.

01 04 -2 -3 -5 08



e = 0.0102" Except

e_6 = 0.009" and

e_8 = 0.009" Water at 68°F

: L = Length (m)

: D = Diameter (m)

: f = Flow at node (gpm)

FIG. 3.2 DATA FOR EXAMPLE

```
          -900.
           01      03      -4      05      -6
          -500.
           01      03      06      07      09
          1500.
           01      03      -7      -8      10
          -800.
           01      02      -9      10
          -1000.
Seventh,   03      01      02      -3
           03      04      05      -2
           04      -6      07      -8      -5
           03      09      10      -7
```

3.7.4 Computer output for the example:

```
Pipe diameter (inches):
     18.0   15.0   18.0  12.0  12.0  10.0  15.0
     18.0   24.0   15.0
Length of pipe (feet):
     1600.  2000.  2400.  1800.  1900.  1300.
     1700.  2000.  1200.  1800.
Relative roughness of pipes:
     .0006  .0007  .0006  .0009  .0009  .0009
     .0007  .0005  .0004  .0007
Number of iterations and errors:
     NCT  = 5,  Sum   = .106E-03
Flowrates in pipes (CFS):
     2.191  .910  2.263  .613  -.272 .228  .647
     .896   2.465  .238
Head losses in pipes (feet)
     .500  .297  .797  .389  .092  .110  .134
     .116  .111  .023
```

Flowrate (GPM):

984.0 408.6 1016.0 275.4 -122.2 102.4 290.6

402.4 1107.0 107.0

Since the computer output confirms the answer given for the problem in the text book Jeppson (1981) page 79, we could conclude that the program is correct.

3.7.5 Modifications in the original program:

There was no choice other than doing modifications in the formats of read and write statements to display in relatively smaller screen to avoid any possible mistakes in reading and to have better presentation of results in a larger number of pipes in a network.

The modifications are as follows:

- Instead of reading D(i) L(i) and E(i) seperately for each pipe from three different very long raws (i from 1 to maximum of 49 in our case) of data, these statements were replaced by a single Read Statement to read the values of D, L & E at a time for each pipe.
- The format of the results were changed to print only five results in a raw to avoid any confusion and overlapping of print out.
- Instead of reading directly, the discharge/ inflow at the nodes, the statement was replaced by two statements to read rate of consumption per house per minute first before it read the number of houses (demand) at each node.

3.7.6 Read statements in the modified program:

The Read statements in the modified program are as follows:

First, Consumption rate per house per minutes (qqq)

Second, NP, NJ, NL, max, unit, error and ΔQ

Third, D, L, E for i = 1 to NP

Fourth, Iflow, NNJ, JN for 1 to NNJ

Fifth, QJ (equivalent to number of houses at junction) of Iflow is greater than zero

Sixth, NNJ, LP (from 1 to NNJ)

3.7.7 Input data for the example solved by the modified program:

The input data for the same example solved by the modified program as follows:

First, 4.0

Second, 10. 07 04 10 00 0,001 0.0000127 0.1

Third, 18.0 1600.0 0.0102
15.0 2000.0 0.0102
18.0 2400.0 0.0102
12.0 1800.0 0.0102
12.0 1900.0 0.0102
10.0 1300.0 0.009
15.0 1700.0 0.0102
18.0 2000.0 0.009
24.0 2000.0 0.0102
15.0 1800.0 0.0102

Fourth, 01 02 01 03
&
Fifth, 500.
01 03 -1 02 04
-75
01 04 -2 -3 -5 08

```
      -225
        01    03    -4    05    -6
      -125.
        01    03    06    07    09
       375
        01    03    -7    -8    10
      -200
        01    02    -9    10
       250
Sixth, 03    01    02    -3
       03    04    05    -2
       04    -6    07    -8     -5
       03    09    10    -7
```

3.7.8 Results from the modified program and conclutions:

The obtained results were same as the results obtained with original program.

Therefore we conclude that the modified program is correct. Herein we will use this tested modified program for all our analysis.

3.8 Computational Problems:

3.8.1 Sub-routine to solve linear system of equations (call statement):

In the original program, the author has used Univac Math-Pack to solve linear system of equations. The major problem and the delay at the initial stage of programming was due to lack of any details about the Univac Math-Pack, which was the key step in the main program.

After trying a number seperate sub-routines, a general sub-routine for GAUSS ELIMINATION WITH PARTIAL PIVOTING was adopted in place of Univac Math-Pack in original program to solve linear system of equations.

3.8.2 Difficulties with SI units:

The program failed to give the results for the example considered when the data was changed into SI unit. Instead reported as SOLUTION NOT FEASIBLE, ANEAR ZERO PIVOT WAS ENCOUNTERED.

Therefore, the English system of units were used in preparing the data for all pipe networks in this project and the results were converted into SI unit as given in each section.

3.8.3 Pipes with negligible flowrates:

Relatively negligible or no flow conditions in a pipe or pipes in a network should be avoided when preparing the schematic network diagram for analysis. Otherwise usually the computer reports as "zero devided by an integer." This is because zero element or elements are encountered as diagonal elements of the matrix resulted from Gauss elimination by partial pivoting.

The best example is area G with schematic network diagram of variant G2

CHAPTER 4

DEFINITIONS, COMMON ASSUMPTIONS AND LIMITATIONS

4.1 Definitions and limitations;

4.1.1 Hydraulic gradient line elevation (H.G.L.) when the pressure head at feeding point is zero H_e^n

For each node, the hydraulic gradient line elevation when the pressure head at feeding point is zero, is computed as follows:

$$H_e^n = H_e^{up} - \Delta H_{up,n} \quad \text{for all n except n = m} \qquad (4.1)$$

$$\text{and } H_e^m = Z_m \quad \text{for} \quad n = m \qquad (4.2)$$

where m = Feeding node no.

n = node no. except feeding node

$\Delta H_{up,n}$ = head loss between upstream node (up) and considered one (n)

4.1.2 Relative pressure head: H_r^n

The relative pressure head H_r^n is defined as follows:

$$H_r^n = H_e^n - Z^n \qquad (4.3)$$

where Z^n = Topo level of considered node

H_e^n = Hydraulic gradient line elevation at node n when feeding pressure is zero.

4.1.3 Standard minimum pressure head (^{h}min.):

Standard minimum pressure head at a node in a network is totally dependent on the need of the consumers.

Guide line given by A.Chernov et al (1969) on standard minimum pressure head to be maintained is as follows:

6m for single storey
12m for second storey
16m for third storey
20m for fourth storey and
25m for fifth storey

4.1.4 Operational pressure head:

Operational pressure head at feeding point H^m_{opr} is defined as follows:

$$H^m_{opr} = \text{abs}\left[\underset{n = 1,N}{\text{Min}}\ H^n_r\right] + {}^{h}\text{min} \quad .. \quad .. \qquad (4.4)$$

where m = Feeding node no.

H^n_r = Relative pressure head at node no. n

N = Total number of nodes in the network

Similarly the operational pressure head at remaining nodes is defined as follows:

$$H^n_{opr} = H^m_{opr} + H^n_r \quad .. \quad .. \quad .. \quad .. \qquad (4.5)$$

where H^n_{opr} = Operational pressure head at remaining nodes

4.1.5 Measure of satisfaction (M.O.S.):

It is a measure to give an idea of how much pressure head is been developed at a given node inaddition to minimum standard pressure head as described above. It is defined as follows:

$$\text{M.O.S.} = \frac{H^n_{opr} - {}^{h}\text{min}}{{}^{h}\text{min}} \quad .. \quad .. \quad .. \qquad (4.6)$$

where H^n_{opr} = Operational pressure head at node n

4.1.6 Average increase in percentage of satisfaction:

It is the percentage ratio between the increase in operational pressure head and standard minimum pressure head is calculated as follows:

$$= \frac{1}{N} \sum_{n=1}^{N} \frac{H^{n}_{opr}}{h_{min}} \times 100 \quad .. \quad .. \quad (4.7)$$

where H^{n}_{opr} = Increase in operational pressure head at node n

4.2 Common assumptions:

4.2.1 Consumption:

The estimated peak consumption rate per house in low pressure zone was used to analysis the pipe network of nigh pressure zone.

According to the investigation carried out by the Department of Water Resources, A.B.U., the peak period of consumption in low pressure zone is between 9.00 a.m. and 10.00 a.m.

Therefore the experiment was carried out from 9.00a.m. to determine the rate of consumption per house in the low pressure zone when the students were not in the Campus.

The areas and its total number of houses in the low pressure zone are as follows:

No. of Houses in area A = 157

No. of Houses in area BZ = 122

No. of Houses in Water Work = 37

90 families in the flats and 180 families in area III were taken as equivalent to 90 houses in other residential areas.

(i.e. one third of No. of families).

Therefore, total No. of House = 406 Houses.

The demand in the academic area in the main campus was assumed to be equivalent to about 10 No. running taps at a time.

From an experiment carried out in low pressure zone, the discnarge from a 75mm tap was estimated as 0.1 l/second.

Therefore, demand in academic area = 10 x 0.1 =1.0 l/s.

From the experiment the average discharge per second from the tank to low pressure zone specified above = 61.8 l/s.

Therefore, Consumption rate per house $= \frac{(61.8 - 1.0\ l/s}{406} = 0.150\ l/s$

Therefore, for the purpose of design, consumption rate/ house is assumed as 0.15 l/s.

4.2.2 Roughness of A. C. pipes:(E)

According to the guidline given by Webber N.B. (1971) pp. 100 the Asbestos cement pipes are considered as smooth pipe

From the table given by Jeppson (1981) pp. 4, the Hassan Williams Coeffients C_{HW} for smooth pipe and pvc pipes are 140 and 150 respectively. On the other hand roughness E for pvc pipe is 0.00021 cm

Therefore, roughness of Asbestos cement pipes $= (0.00021 \times \frac{150}{140}) = 0.000225$ Cm.

Hence, for our analysis roughness of Asbestos cement pipe (A.C) is taken as 0.00025 Cm.

4.2.3 Minor losses in the pipe network: ΔL

To determine the equivalent pipe length for valves in the network, the following formulae is being used:

$$\Delta L = \frac{K_L D}{f} \quad .. \quad .. \quad .. \quad .. \quad (4.7)$$

where K_L = Loss co-efficient for valves

f = Friction factor

For the calculation of minor losses due to valve, the value of K_L is taken as ten as given by Jeppson (1981) pp. 50.

Therefore, the pipe lengths of those containing valves in the pipe network is computed as increasing the pipe length by its calculated value of ΔL.

4.2.4 Frictional factor f

The friction factor f is being read out from Moody diagram for the known value of Reynold number and relative roughness.

The Reynold number Re is defined as:

$$Re = \frac{VD}{\nu} = \frac{4}{\pi D \nu} Q \quad .. \quad .. \quad (4.8)$$

Relative roughness is defined as

$$= \frac{E}{D} \quad .. \quad .. \quad .. \quad .. \quad (4.9)$$

4.2.5 Frictional head losses h_f:

The most foundamentally sound method for computing friction head loss is by means of Darcy-Weisbach equation $h_f = f \frac{L}{D} \frac{V^2}{2g}$

$$h_f = \frac{8f}{\pi^2 g} \frac{Q^2}{D^5} L \quad .. \quad (4.10)$$

4.2.6 Kinematic viscosity (ν):

Assumed tempt of water in the pipe = $20^{\circ}C$ by Jeppson (1981) pp. 4. The kinematic viscosity (ν) = 1.007×10^{-6} m^2/s at $20^{\circ}C$

4.2.7 Topo level of nodes:

Topo level of each nodes are tabulated in its relevant sections, were obtained from topo sheets scalled at 1:2400.

CHAPTER 5

APPLICATION OF LINEAR THEORY METHOD TO A.B.U. DISTRIBUTION NETWORKS

Distribution networks of all residential areas were considered seperately to carry out hydraulic computations using Fortran Computer program described in chapter 3 (program is given in Appendix 'C')

5.1 Preparation of pipe network diagram:

Although there was no restriction on pipe lengths and external flow at nodes, a maximum limit of 300m for pipe lengths and 10 houses for a node were generally adopted, except where the limit of 10 houses was not followed in cases where point loads (represents tree type or open distribution mains) were encountered.

5.2 Computer run:

The input data was prepared for each pipe network diagram of the problem formulated in each area. The program was run and the results were obtained.

5.3 Computer input/output:

Computer input data are prepared seperately for each residential area .

From the computer run, the two most important results obtained as output are as follows: (sub-sec. 3.7)

(a) Friction head losses in pipe (ft), converted into (m)

(b) Flowrate in pipes (ft^3/s) converted into m^3/s.

5.4 Computation time:

In general, the computation real time was 2 minutes and 50 sec. for all areas except for area E for which 2 mts. and 05 sec. was taken by personal computer IBM model XT.

5.5 Residential area 'C'

5.5.1 Formulation of problems:

Hydraulic computation of pipe networks were carried out seperately in order to

(a) estimate the required operational pressure head at feeding points and to determine the one with the highest satisfaction factor.

(b) Influence of minor losses on distribution of pressure head.

5.5.2 Common data:

A common diagram of the pipe network shown in Fig. 5.1 gives pipe Nos., junction Nos. and loop Nos.

NP = 42 (No. of pipes in the networks)

NJ = 37 (No. of junction in the network)

NL = 06 (No. of loops in the network)

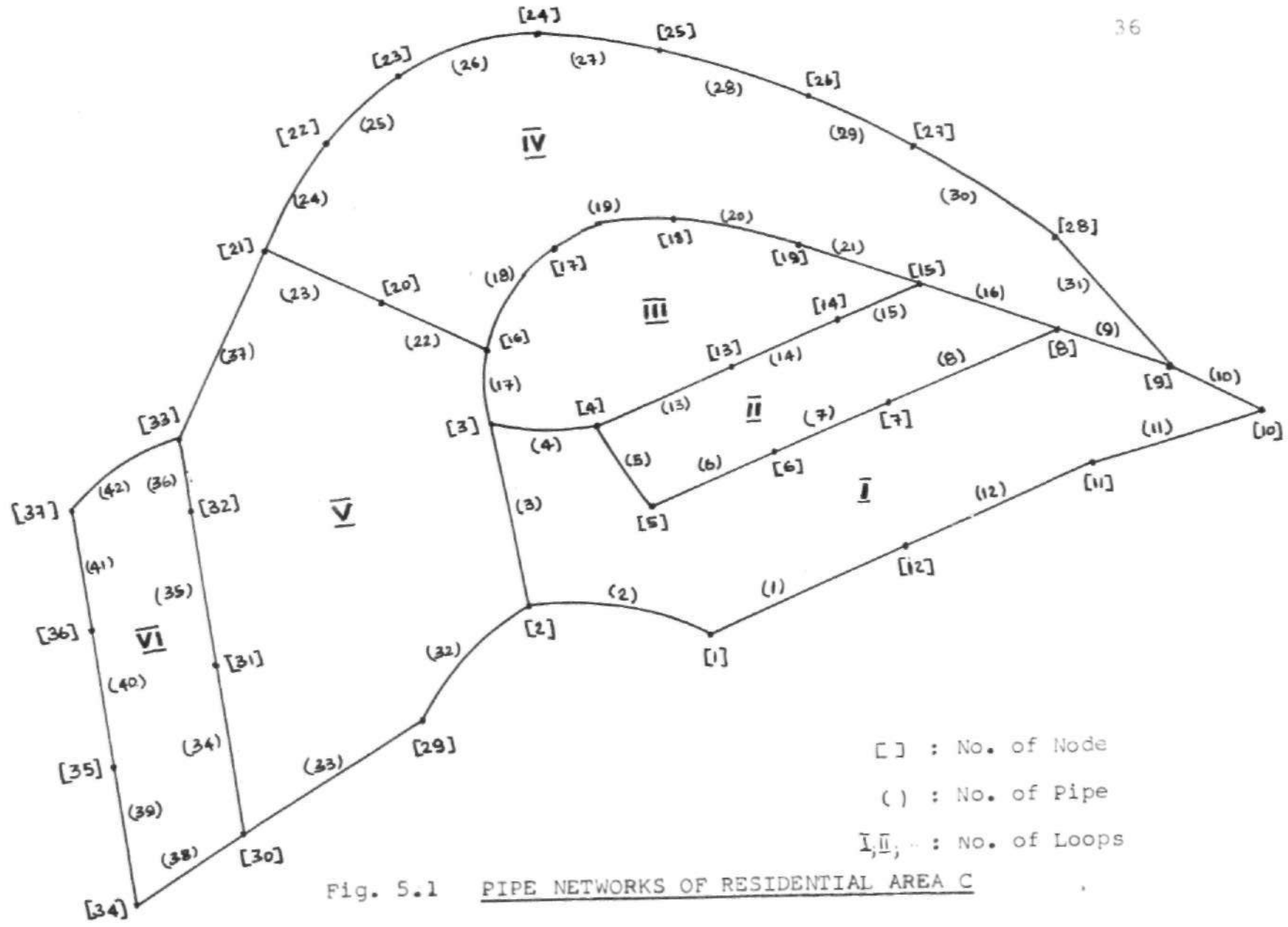
Total No. of houses = 129

Lengths and diameter of pipes and external flows are shown in Fig. 5.2.

5.5.3 Solution of problem A

5.5.3.1 Data for hydraulic computation:

In problem A, the computations were carried out seperately with three different feeding points. One of the feeding point was chosen as the closest node from the proposed central over head tank while the other two are the existing supply points namely Kaduna State Water Supply and the supply through area 'C' from A.B.U. Water Works.



[ ] : No. of Node

( ) : No. of Pipe

$\overline{I}, \overline{II}, \ldots$: No. of Loops

Fig. 5.1 PIPE NETWORKS OF RESIDENTIAL AREA C

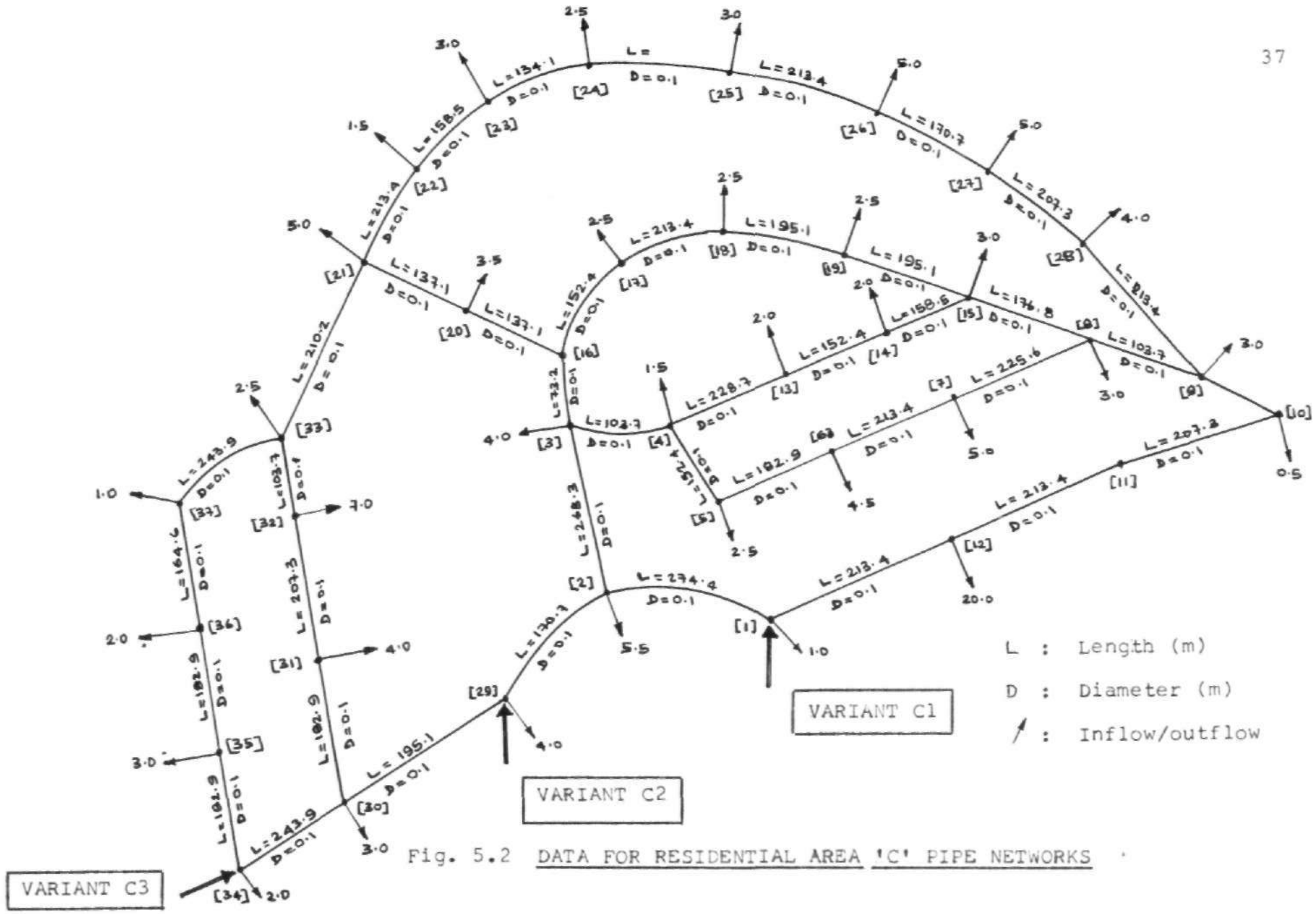


Fig. 5.2 DATA FOR RESIDENTIAL AREA 'C' PIPE NETWORKS

The three variants are as follows:

Variant C1: Network with a feeding point chosen as the closest node from the central over head tank:

Feeding point is node No. 1

Demand at node No. 1 is equivalent to 1.0 house

Therefore, Net external inflow at the feeding point is equivalent to (129-1)

Houses = 128 Houses

Variant C2: Network with the supply through area 'G' from A.B.U. Water Works:

Feeding point is node No. 29

Demand at node No. 29 is equivalent to 4 houses

Therefore, Net external inflow at the feeding point is equivalent to (129-4)

= 125 houses

Variant C3: Network with the supply from Kaduna State Water Board:

Feeding point is node No. 34

Demand at node No. 34 is equivalent to 2 houses

Therefore, Net external inflow at the feeding point is equivalent to (129-2)

= 127 houses.

5.5.3.2 Results and Analysis:

Flowrate in pipes for each system of feedings obtained from computer run are as shown in Figs. 5.3, 5.4 and 5.5. Similarly the frictional head losses in pipes are tabulated in Table 5.1.

Operational pressure heads at feeding points: Knowing the topo level of the up stream node and the frictional head losses in the pipe, the hydraulic gradient line elevation, and the relative pressure head at nodes are computed and results are also shown in Table 5.1.

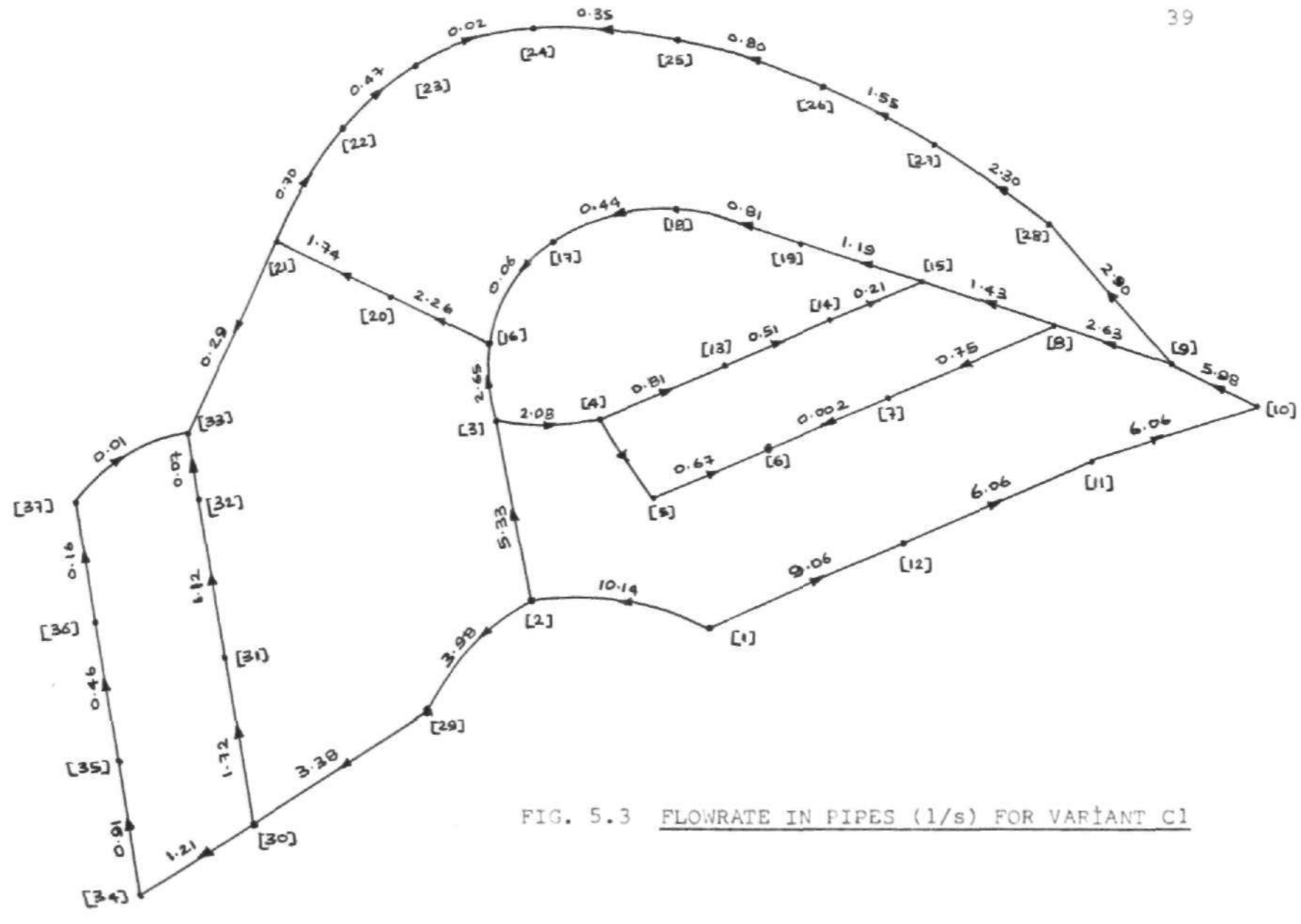


FIG. 5.3 FLOWRATE IN PIPES (l/s) FOR VARIANT C1

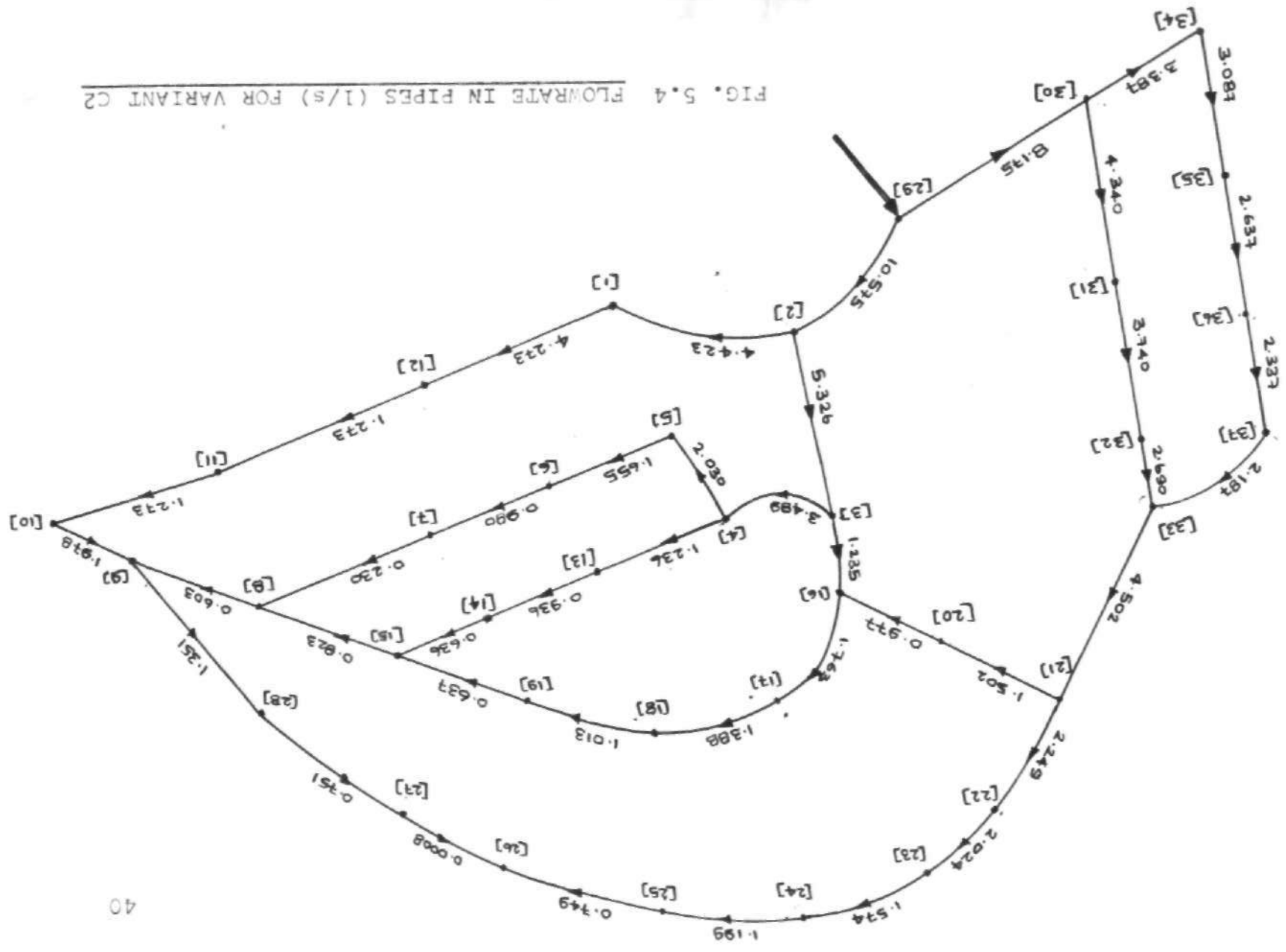


FIG. 5.4 FLOWRATE IN PIPES (l/s) FOR VARIANT C2

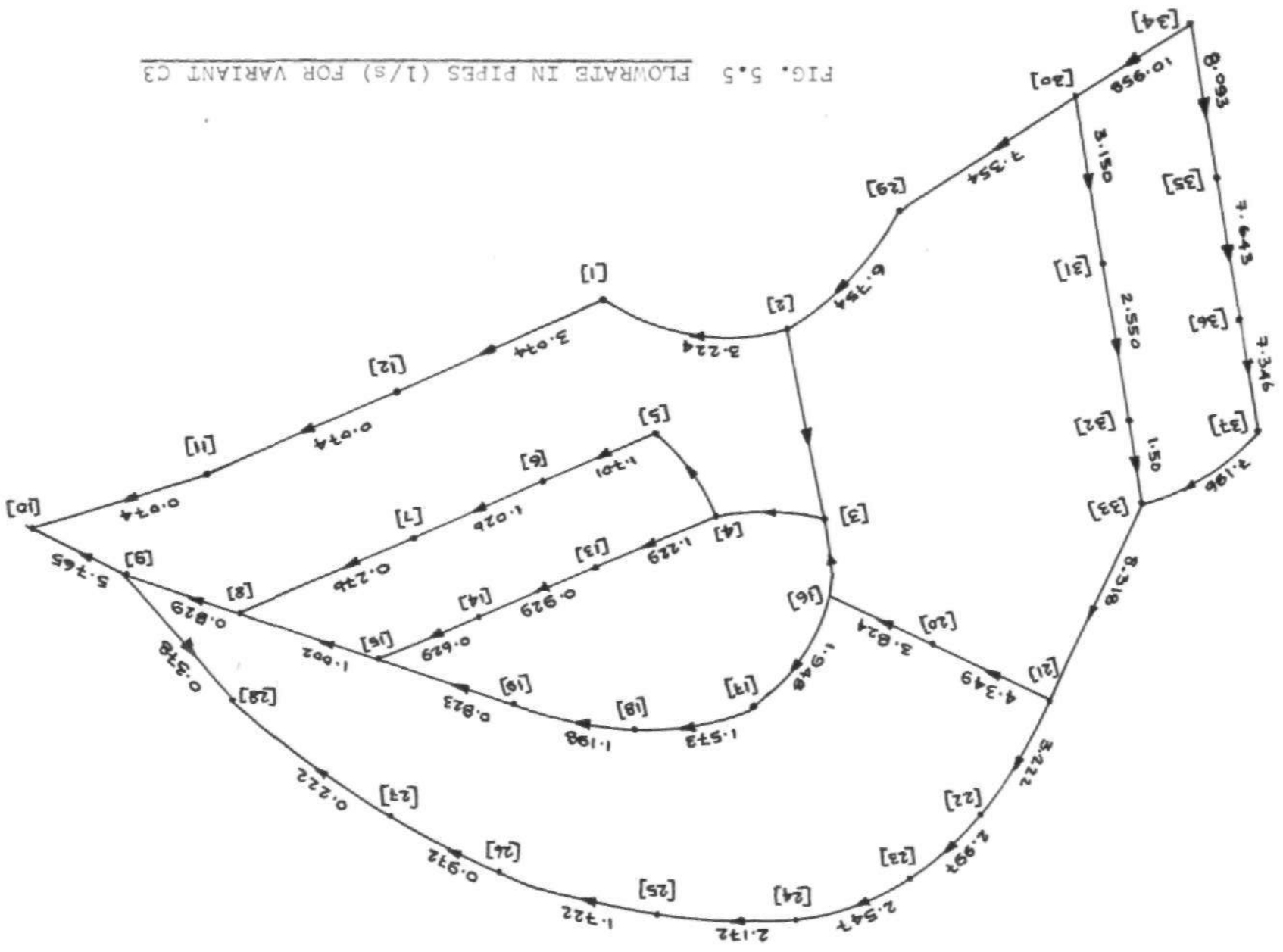


FIG. 5.5 FLOWRATE IN PIPES (1/s) FOR VARIANT C3

TABLE 5.1: RELATIVE PRESSURE HEADS AT NODES

JN No.	Pipe No.	Topo Level	H.G.L. Elevation when the pressure head at feeding point is zero H_e^n (m)			Frict. head loss in pipe $H_{up,n}$ (m)			Relative pressure head - that is when feeding pressure head is zero H_L^n (m)		
			C1	C2	C3	C1	C2	C3	C1	C2	C3
1	2	3	4	5	6	7	8	9	10	11	12
01		693.903	693.902	693.406	685.807				0.000	-0.496	8.095
	02					2.727	0.618	0.353			
02		696.341	691.175	694.026	686.160				-5.166	-2.315	-10.181
	03					0.844	0.842	0.253			
03		694.146	690.331	693.184	685.907				-3.815	-0.962	-8.239
	04					0.062	0.153	0.157			
04		693.293	690.269	693.031	685.750				-3.024	-0.262	-7.543
	05					0.027	0.87	0.90			
05		693.293	690.242	692.944	685.660				-3.051	-0.349	-7.633
	06					0.015	0.073	0.076			
06		690.804	690.227	692.871	685.584				-0.657	1.987	-5.300
	07					0.002	0.034	0.037			
07		688.780	690.227	692.837	685.547				1.447	4.057	-3.233
	08					0.023	0.003	0.004			

1	2	3	4	5	6	7	8	9	10	11	12
08		688.034	690.250	692.834	685.543				2.216	4.800	-2.491
	09					0.093	0.007	0.012			
09		687.500	690.343	692.827	685.531				2.843	5.327	-1.969
	10					0.259	0.015	0.00001			
10		686.357	690.602	692.842	685.531				3.245	5.485	-1.826
	11					0.819	0.052	0.006			
11		686.451	691.421	692.894	685.537				4.970	6.443	-0.914
	12					0.843	0.054	0.006			
12.		689.787	692.264	692.948	685.543				2.477	3.161	-4.244
	-					-	-	-			
13		691.463	690.243	692.976	680.696				-1.220	1.513	-5.767
	14					0.008	0.022	0.022			
14		690.549	690.235	692.954	685.674				-0.314	2.405	-4.875
	15					0.013	0.012	0.012			
15		690.030	690.222	692.942	685.662				0.192	2.912	-4.368
	-					-	-	-			
16		694.024	690.264	693.167	685.929				-3.760	-0.857	-8.095

1	2	3	4	5	6	7	8	9	10	11	12
	18					0.004	0.068	0.091			
17		693.902	690.268	693.008	685.848				-3.634	-0.803	-8.054
	19					0.008	0.062	0.078			
18		693.598	690.276	693.037	685.770				-3.322	-0.561	-7.828
	20					0.022	0.033	0.044			
19		695.122	690.298	693.004	685.726				-4.824	-2.118	-9.396
	-					-	-	-			
20		693.598	690.170	693.182	685.930				-3.428	-0.416	-7.668
	23					0.059	0.046	0.300			
21		692.683	690.111	693.228	686.230				-2.572	0.545	-6.453
	24					0.019	0.145	0.274			
22		691.845	690.092	693.083	685.956				-1.753	1.238	-5.889
	25					0.007	0.090	0.179			
23		691.768	690.085	692.993	685.777				-1.683	1.225	-5.991
	26					0.001	0.049	0.114			
24		692.378	690.986	692.944	685.663				-2.292	0.566	-6.715

1	2	3	4	5	6	7	8	9	10	11	12
	27					0.006	0.048	0.137			
25		693.293	690.092	692.896	685.526				-3.201	-0.397	7.767
	28					0.024	0.021	0.091			
26		694.360	690.116	692.875	685.435				-4.244	-1.485	-8.925
	29					0.061	0.0001	0.027			
27		694.207	690.177	692.875	685.408				-4.030	-1.454	-8.799
	30					0.147	0.021	0.018			
28		689.878	690.324.	692.896	685.390				-0.446	2.996	-4.488
	-					-	-	-			
29		695.854	690.856	695.854	686.979				-4.998	0.00	-8.875
	33					0.273	1.318	1.090			
30		692.988	690.583	694.536	688.069				-2.405	1.548	-4.919
	34					0.078	0.398	0.226			
31		693.415	690.505	694.138	687.843				-2.910	0.723	-5.572
	35					0.048	0.347	0.176			
32		693.293	690.457	693.791	687.667				-2.836	0.498	-5.626
	36					0.003	0.097	0.035			

1	2	3	4	5	6	7	8	9	10	11	12
33		692.348	690.460	693.694	687.632				-1.894	1.346	-4.716
	-					-	-	-			
34		690.854	690.527	694.194	690.854				-0.327	3.340	0.00
	39					0.026	0.218	1.213			
35		690.500	690.501	693.976	689.641				-0.001	3.476	-0.859
	40					0.008	0.165	1.095			
36		690.244	690.493	693.811	688.546				0.249	3.567	-1.698
	41					0.101	0.120	0.917			
37		689.939	690.392	693.691	687.679				0.453	3.752	-2.260

In every system of feedings the minimum relative pressure heads were obtained only at node No. 2 as follows:

For Variant C1, $H_r^2 = -5.166m$

Variant C2, $H_r^2 = -2.315m$

Variant C3, $H_r^2 = -10.181m$

Therefore, the operational pressure heads to be maintained at each feeding points are as follows: (Eq. 4.4)

For the variant C1, $H_{opr}^1 = 12.00 + 5.166 = 17.166m$

the variant C2, $H_{opr}^{29} = 12.00 + 2.315 = 14.315m$

the variant C3, $H_{opr}^{34} = 12.00 + 10.181 = 22.181m.$

Measure of satisfaction (M.O.S.):

Operational pressure heads at nodes were computed for each system of feeding independently as shown in Table 5.2

Knowing the differences between the operational pressure head and standard minimum pressure head at each node the measure of satisfactions as computed in Eq. (4.6) are also shown in Table 5.2.

The computed average percentages of satisfaction for each system of feedings are as follows:

for Variant C1 = 30.7%

Variant C2 = 30.6%

Variant C3 = 38.2%

Location of the proposed central over head tank:

The location of the proposed central over head tank is being selected as high as possible within the areas under consideration with a view to reduce the total cost of construction of the tank.

The topo level of the location = 698.171m.

Design of supply lines for each variant:

Total No. of houses = 129

Assumed diameter of pipe (D) = 0.2m

Fig. 5.6 shows the feeding points of different variants

For Variant C1, Length of supply line L_1 = 110m

Variant C2, " " " " L_2 = 635m

Variant C3, " " " " L_3 = 890m

The required rate of supply at each feeding point Q

= 129 x 0.15 1/s = 19.35 x $\frac{-3}{10}$ m^3/s.

TABLE 5.2 PERCENTAGE OF SATISFACTION AT DIFFERENT NODES FOR ALL THREE VARIANTS OF AREA 'C'

JN No.	Operational pressure head H^n_{opr} (m)			$H_{opr} - h_{min}$ (m)			M. O. S. (percentage)		
	C1	C2	C3	C1	C2	C3	C1	C2	C3
1	2	3	4	5	6	7	8	9	10
01	17.166	13.819	14.086	5.166	1.819	2.086	43.0	15.2	17.4
02	12.000	12.000	12.000	0.000	0.000	0.000	0.0	0.0	0.0
03	13.351	13.353	13.942	1.351	1.353	1.942	11.3	11.3	16.2
04	14.142	14.053	14.638	2.142	2.053	2.638	17.9	17.1	22.0
05	14.115	13.966	14.548	2.115	1.966	2.548	17.7	16.4	21.3
06	16.509	16.302	16.881	4.509	4.302	4.881	37.6	35.9	40.7
07	18.613	18.372	18.948	6.613	6.372	6.948	55.1	53.1	57.9
08	19.382	19.115	19.690	7.382	7.115	7.690	61.5	59.3	64.1
09	20.009	19.642	20.212	8.009	7.642	8.212	67.8	63.7	68.5
10	20.411	19.800	20.355	8.411	7.800	8.355	70.1	65.0	69.7
11	22.136	20.758	21.267	10.136	6.758	9.267	84.5	73.0	77.3
12	19.643	17.476	17.937	7.643	5.476	5.937	63.7	45.7	49.5

1	2	3	4	5	6	7	8	9	10
13	15.946	15.828	16.414	3.946	3.828	4.414	32.9	31.9	36.8
14	16.852	16.720	17.306	4.852	4.720	5.306	40.5	39.6	44.2
15	17.358	17.227	17.813	5.358	5.227	5.813	44.7	43.6	48.5
16	13.406	13.458	14.086	1.406	1.458	2.086	11.7	12.2	17.4
17	13.532	13.512	14.127	1.532	1.512	2.127	12.8	12.6	17.8
18	13.844	13.754	14.353	1.844	1.754	2.353	15.4	14.6	19.6
19	12.342	12.197	12.785	0.342	0.197	0.785	2.9	1.7	6.6
20	13.738	13.899	14.513	1.738	1.899	2.513	14.5	15.9	20.9
21	14.594	14.860	15.728	2.594	2.860	3.728	21.6	23.9	31.1
22	15.413	15.553	16.292	3.413	3.553	4.292	28.5	29.6	35.8
23	15.483	15.540	16.190	3.483	3.540	4.190	29.6	29.5	34.9
24	14.874	14.881	15.466	2.874	2.881	3.466	23.9	24.0	28.9
25	13.965	13.918	14.414	1.965	1.918	2.414	16.4	16.0	20.1
26	12.922	12.830	13.256	0.922	0.830	1.256	7.7	6.9	10.5
27	13.136	12.861	13.382	1.136	0.861	1.382	9.5	7.2	11.5
28	16.720	17.211	17.693	4.720	5.211	5.693	39.4	43.5	47.5

1	2	3	4	5	6	7	8	9	10
29	12.168	14.315	13.306	0.168	2.315	1.306	1.4	19.3	10.9
30	14.761	15.863	17.262	2.761	3.863	5.262	13.0	32.2	43.9
31	14.256	15.038	16.509	2.256	3.038	4.609	18.8	25.3	38.4
32	14.330	14.813	16.555	2.330	2.813	4.555	19.4	23.5	37.9
33	15.272	15.661	17.465	3.272	3.661	5.465	27.3	30.5	95.6
34	16.839	17.655	22.181	4.839	5.655	10.181	40.4	47.7	84.9
35	17.165	17.791	21.322	5.165	5.791	9.322	43.1	48.3	77.7
36	17.415	17.882	20.483	5.415	5.862	8.483	45.2	49.0	70.7
37	17.619	18.067	19.921	5.619	6.067	7.921	46.9	50.6	66.0
						The average percentage of satisfaction	30.7	30.6	38.2

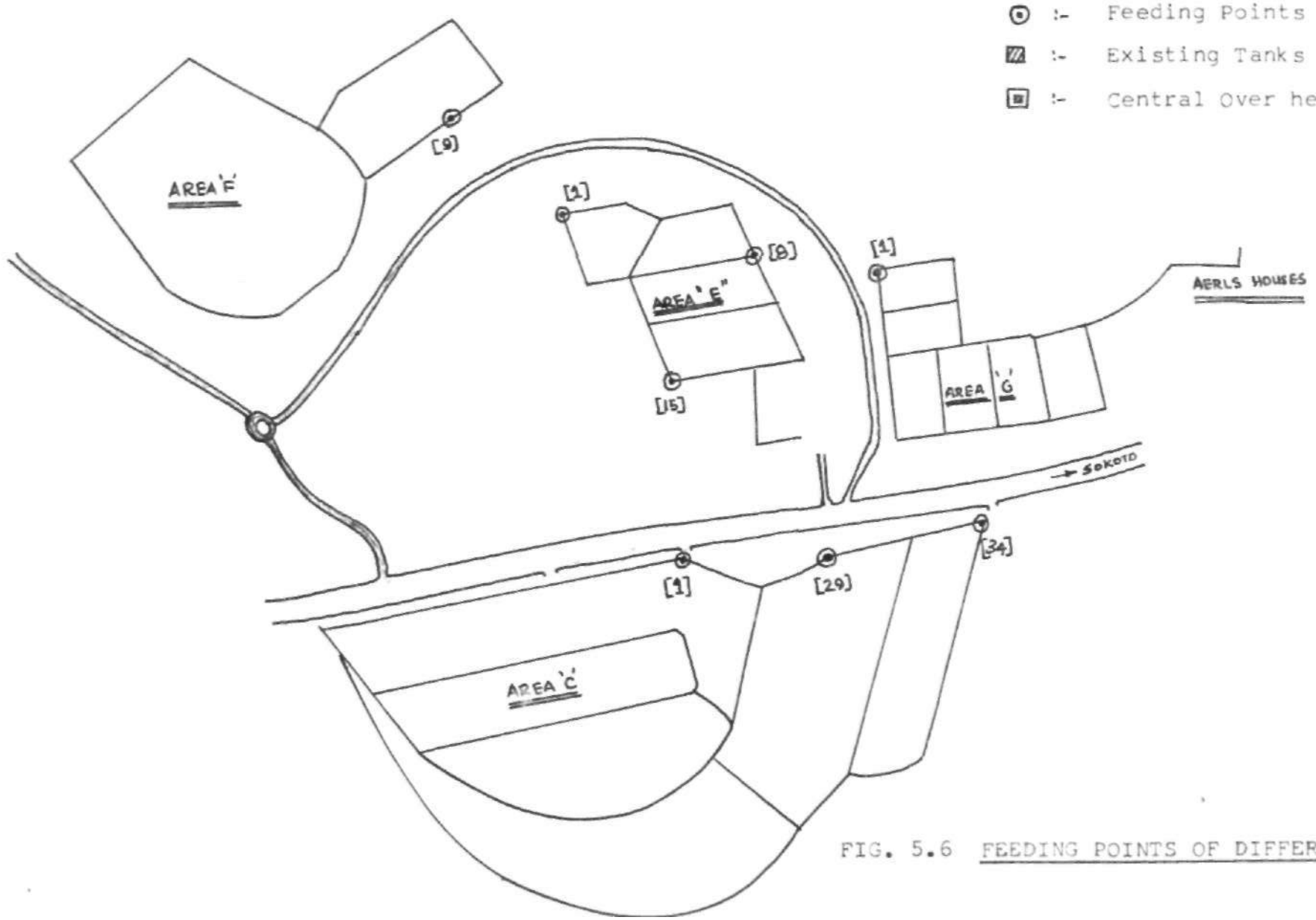


FIG. 5.6 FEEDING POINTS OF DIFFERENT VARIANTS

(a) The Reynold number Re

Eq. (4.8)

$$Re = \frac{4}{\pi D \mu} \cdot Q$$

$$Re = \frac{4 \times 19.35 \times 10^{-3} \quad m^3/s}{\times 0.2 \times 1.007 \times 10^{-6} \quad m^3/s}$$

$$Re = 1.22 \times 10^{5}$$

(b) The relative roughness

Eq. (4.9)

$$= \frac{E}{D} = .0000125$$

From Moody diagram the friction factor

$$f = 0.017$$

(c) The frictional head loss h_f

Eq. (4.10)

$$h_f = \frac{8(1.7 \times 10^{-2}) \times (19.35)^2 \times 10^{-6}}{(3.14)^2 \times 9.8 \times (0.2)^5} \quad L$$

$$h_f = 0.0016 \quad L$$

Therefore, for Variant C1, h_f = 0.176m

Variant C2, h_f = 1.016m

Variant C3, h_f = 1.424m

As the calculated frictional head losses in supply lines are small as acceptable, the diameter of the supply lines are chosen as 0.2m.

Computation of the tank height:

(a) Variant C1:

Minimum pressure head required at feeding point = 17.166m

Topo level at feeding point = 693.902m

Hence, H.G.L. elevation at feeding point

= 693.902 + 17.166 = 711.068m

Frictional head losses on supply line

= 0.176m

Hence, H.G.L. elevation required at bottom of the tank = 711.068 + 0.176

= 711.244m

Topo level at the bottom of the tank = 698.171

Hence, pressure head required at bottom of the tank = 711.244 -698.171

= 13.073m

(b) Variant C2:

At feeding point;

Minimum pressure head required = 14.315m

Topo level = 695.854m

Hence, H.G.L. elevation required = 695.85 + 14.315

= 710.169m

Frictional head losses in supply line = 1.016m

At the bottom of the tank;

H.G.L. elevation required = 710.169 +1.016

= 711.185m

Topo level = 698.171m

Hence, pressure head required = 711.185 - 698.171

= 13.014m.

(c) Variant C3:

At feeding point:

Minimum pressure head required = 22.181m

Topo level = 690.854m

Hence, H.G.L.elevation required = 690.854 + 22.181

= 713.035m

Friction head losses in supply line = 1.424m

At the bottom of the tank:

H.G.L. elevation required = 713.035 + 1.424
= 714.459m

Topo level = 698.171m

Hence, pressure head required = 714.459 - 698.171

= 16.288m

Table 5.3: Comparison of height of water surface and percentage of satisfaction for all three variants to satisfy minimum pressure head requirements at all nodes:

	Variant C1	Variant C2	Variant C3
Height of water surface (m)	13.07	13.01	16.29
Average percentage of satisfaction	30.7	30.6	38.2

The average percentage of satisfaction is a function of height of water surface. The computed percentage of satisfaction listed above are for different heights of the tank. Therefore, comparison does not reveal much unless the computation of percentage of satisfaction for each system of feeding is carried out with a constant height of water surface.

Comparison of the system of feedings:

For the purpose of comparison, a common source of supply (constent height) was assumed. The height of the common tank was chosen as the height of the tank in variant C^, because this is the heightest tank in all the three variants which could satisfy the minimum standard pressure head requirement at every node in all three variants.

The chosen height of the tank = 16.29m

(a) Determination of operational pressure head at feeding points:

Variant C1:

H.G.L. elevation at bottom of the tank = 714.459m

Friction head losses in supply line between feeding point and the tank = 0.176m

Hence, H.G.L. elevation at feeding point = 714.459-0.176

= 714.283m

Hence, Operational pressure head at feeding point

= 714.283 - 693.902

= 20.381m

Variant C2:

H.G.L. elevation at bottom of the tank = 714.459m

The frictional head loss in supply line between the feeding point and the tank = 1.016m

Hence, H.G.L. elevation at feeding point = 714.459 - 1.016

= 713.443m

Hence Operational pressure head at feed point

= 713.443 - 695.854

= 17.589m

(b) Computation of percentage of satisfaction:

Knowing the corresponding pressure heads at the feeding points of the variants C1 and C2, the new percentage of satisfactions could be computed as follows:

Variant C1:

The average increase in pressure head at every node = 20.381-17.166 = 3.215m

Hence, increase in percentage of satisfaction

$$= \frac{3.215}{12} = 26.8\%$$

Hence, the new percentage of satisfaction

$$= 30.7 + 26.8 = 57.5\%$$

Variant C2:

The average increase in pressure head at nodes

$$= 17.589-14.315 = 3.274\text{m}$$

Hence, increase in percentage of satisfaction

$$= \frac{3.274}{12} = 27.3\%$$

Hence, the new percentage of satisfaction =

$$= 30.6 + 27.3 = 57.9\%$$

Therefore theoritically system with variant C2 will have the maximum percentage of satisfaction out of the three variants considered. This again may or may not be economical compared to others.

Now the lengths of supply lines were measured and compared for economy.

Table 5.4 Comparison of Lengths of supply lines and percentage of satisfactions:

	Variant C1	Cariant C2	Variant C3
Average percentage of satisfaction	57.5	57.9	38.2
Length of supply lines (m)	110.0	636.0	890.0

The length of the supply line of the variant C2 is about 6 times of the length of the supply line of the variant C1. On the other hand the difference on the percentage of satisfaction is small as neglegible.

5.5.3.3 Conclutions:

The best recommended feeding point for the residential area 'C' is the node No. 1. The required operational pressure head is 17.17m.

5.5.4 Solution of problem B

5.5.4.1 Data for hydraulic computations:

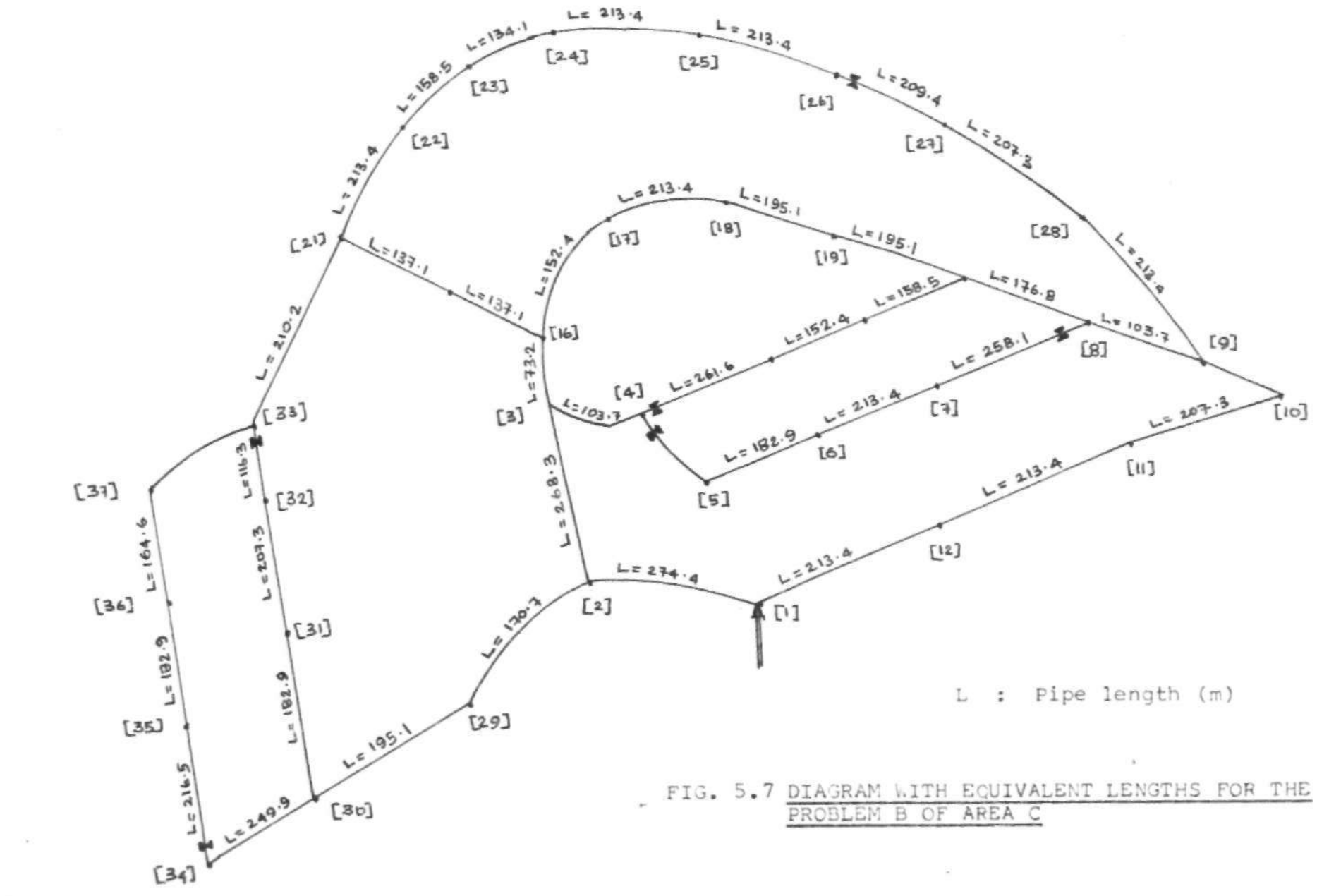
The first step is to find the equivalent pipe lengths for those pipes containing the valves in the network. For this purpose network with variant C1 . has been considered.

The computed values of ΔL for pipes containing valves in the network are as follows:

Table 5.5 Equivalent pipe lengths for valves in area 'C':

Pipe No.	5	8	10	13	29	36	39
Q (cfs)	0.0311	0.0223	0.1778	0.024	0.0462	0.0021	0.027
Q x 10^4 (m^3/s)	8.8	6.3	0.50	6.8	13.1	0.59	7.6
Re x 10^3	11.12	7.96	0.63	8.60	16.56	0.75	9.61
f	0.029	0.031	0.0195	0.0305	0.026	0.08	0.08
ΔL Eq.(4.7) (m)	34.48	32.26	51.28	32.79	38.46	12.5	12.5

The Computed equivalent lengths of pipes are as shown in Fig. 5.7. All other data are remain same as for variant C1 in the problem A.



L : Pipe length (m)

FIG. 5.7 DIAGRAM WITH EQUIVALENT LENGTHS FOR THE PROBLEM B OF AREA C

5.5.4.2 Results and Analysis:

As in problem A, the relative pressure heads respect to feeding point have been computed. The results are tabulated in Tab: 5.5, along with the corresponding results of variant C1 of the problem A (i.e. without valves).

Errors in operational pressure head at nodes:

The errors in operational pressure head at nodes due to minor losses in the network are computed as follows:

$$\text{Error} = \frac{\left[H^{n}_{opr}\right]_{A} - \left[H^{n}_{opr}\right]_{B}}{H^{n}_{opr_{A}}} \quad .. \quad .. \quad .. \quad (5.1)$$

where $\left[H^{n}_{opr}\right]_{A}$ = Operational pressure head at node 'n' in variant C1 of problem A.

$\left[H^{n}_{opr}\right]_{B}$ = Operational pressure head at node 'n' in problem B.

The computed percentage of error in operational pressure head at nodes are as shown in Tab: 5.6.

Flowrates in pipes from computer analysis are shown in Fig. 5.8.

5.1.1.3 Conclutions:

The maximum influence of the minor losses on distribution of pressure head is so small as 1.9%, could be neglected in the analysis of pipe network.

TABLE 5.6: PERCENTAGE OF ERROR IN DISTRIBUTION OF PRESSURE HEAD IN NODES DUE TO MINOR LOSSES FOR PROBLEM B OF GROUP C

JN No.	Pipe No.	Topo Level	H.G.L. elevation when the pressure head at feeding point is zero H_e^n		Friction head losses in pipe (m)		Relative pressure head H_r^n		Diff. in pressure head (m)	Operational pressure head	Percentage of error in dist. of pressure
			Prob. A Cl	Prob. B	Prob. A Cl	Prob. B	Prob. A Cl	Prob. B			
1	2	3	4	5	6	7	8	9	10	11	12
01		693.902	693.902	693.902			0.00	0.000	0.000	11.166	0.0
	02				2.727	2.760					
02		696.341	691.175	691.142			-5.166	-5.199	-0.033	6.000	0.55
	03				0.844	0.927					
03		694.146	690.331	690.215			-3.815	-3.931	-0.116	7.351	1.58
	04				0.062	0.068					
04		693.293	690.269	690.147			-3.024	-3.146	-0.122	8.142	1.50
	05				0.027	0.0360					
05		693.293	690.242	690.111			-3.051	-3.182	-0.131	8.115	1.61
	06				0.015	0.0170					

1	2	3	4	5	6	7	8	9	10	11	12
06		690.884	690.227	690.094			-0.657	-0.790	-0.133	10.509	1.27
	07				0.0002	0.003					
07		688.780	690.227	690.091			1.447	1.311	0.136	12.613	1.08
	08				0.023	0.023					
08		688.034	690.250	690.114			2.216	2.08	0.136	13.382	1.02
	09				0.093	0.087					
09		687.500	690.343	690.201			2.843	2.701	0.142	13.867	1.02
	10				0.259	0.449					
10		686.357	690.602	690.650			4.245	4.293	-0.048	15.411	0.31
	11				0.819	0.803					
11		686.451	691.421	691.453			4.970	5.002	-0.032	16.136	0.20
	12				0.843	0.827					
12		689.787	692.264	692.280			2.477	2.493	-0.016	13.643	0.12
	-				-	-					
13		691.463	690.243	690.112			-1.220	-1.351	-0.131	9.946	1.32

1	2	3	4	5	6	7	8	9	10	11	12
	14				0.008	0.010					
14		690.549	690.235	690.102			-0.314	-0.447	-0.133	10.852	1.23
	15				0.013	0.003					
15		690.030	690.222	690.099			0.192	0.069	0.123	11.358	1.08
	-				-	-					
16		694.024	690.264	690.141			-3.760	-3.883	-0.123	7.406	1.66
	18				0.004	0.004					
17		693.902	690.268	690.145			-3.634	-3.757	-0.123	7.532	1.63
	19				0.008	0.009					
18		693.598	690.276	690.154			-3.322	-3.444	-0.122	7.844	1.56
	20				0.022	0.023					
19		695.122	690.296	690.177			-4.824	-4.945	-0.121	6.342	1.91
	-				-	-					
20		693.598	690.170	690.033			-3.428	-3.565	-0.137	7.738	1.77
	23				0.059	0.071					
21		692.683	690.111	689.962			-2.572	-2.721	-0.149	11.017	1.35

1	2	3	4	5	6	7	8	9	10	11	12
	24				0.019	0.017					
22		691.845	690.092	689.845			-1.753	-1.900	-0.147	11.019	1.33
	25				0.007	0.006					
23		691.768	690.085	689.939			-1.683	-1.829	-0.146	11.02	1.32
	26				0.001	0.001					
24		692.378	690.066	689.940			-2.292	-2.438	-0.146	11.02	1.32
	27				0.006	0.007					
25		693.293	690.092	689.947			-3.201	-3.346	-0.145	11.021	1.32
	28				0.024	0.026					
26		694.360	690.116	689.972			-4.244	-4.388	-0.144	11.022	1.31
	29				0.061	0.078					
27		694.207	690.177	690.05			-4.030	-4.157	-0.127	11.039	1.15
	30				0.147	0.152					
28		689.878	690.324	690.202			0.446	0.324	0.122	11.288	1.08
	-				-	-					
29		695.854	690.856	690.887			-4.998	-4.967	0.031	11.197	0.28
	33				0.273	0.242					

1	2	3	4	5	6	7	8	9	10	11	12
30		690.908	690.583	690.645			-2.405	-2.343	0.062	11.22	0.55
	34				0.078	0.069					
31		693.415	690.505	690.576			-2.911	-2.839	0.071	11.237	0.63
	35				0.018	0.035					
32		693.293	690.457	690.541			-2.836	-2.752	0.084	11.250	0.75
	36				0.003	0.002					
33		692.348	690.454	690.543			-1.894	-1.805	0.089	11.255	0.79
	-				-	-					
34		690.854	690.527	690.598			-0.327	-0.256	0.071	11.237	0.63
	39				0.026	0.024					
35		690.500	690.501	690.574			0.001	0.074	0.073	11.239	0.65
	40				0.008	0.005					
36		690.244	690.493	690.563			0.249	0.319	0.070	11.242	0.68
	41				0.101	0.003					
37		689.939	690.392	690.566			0.453	0.627	0.174	11.34	1.53

The maximum error in distribution of pressure head = 1.91%

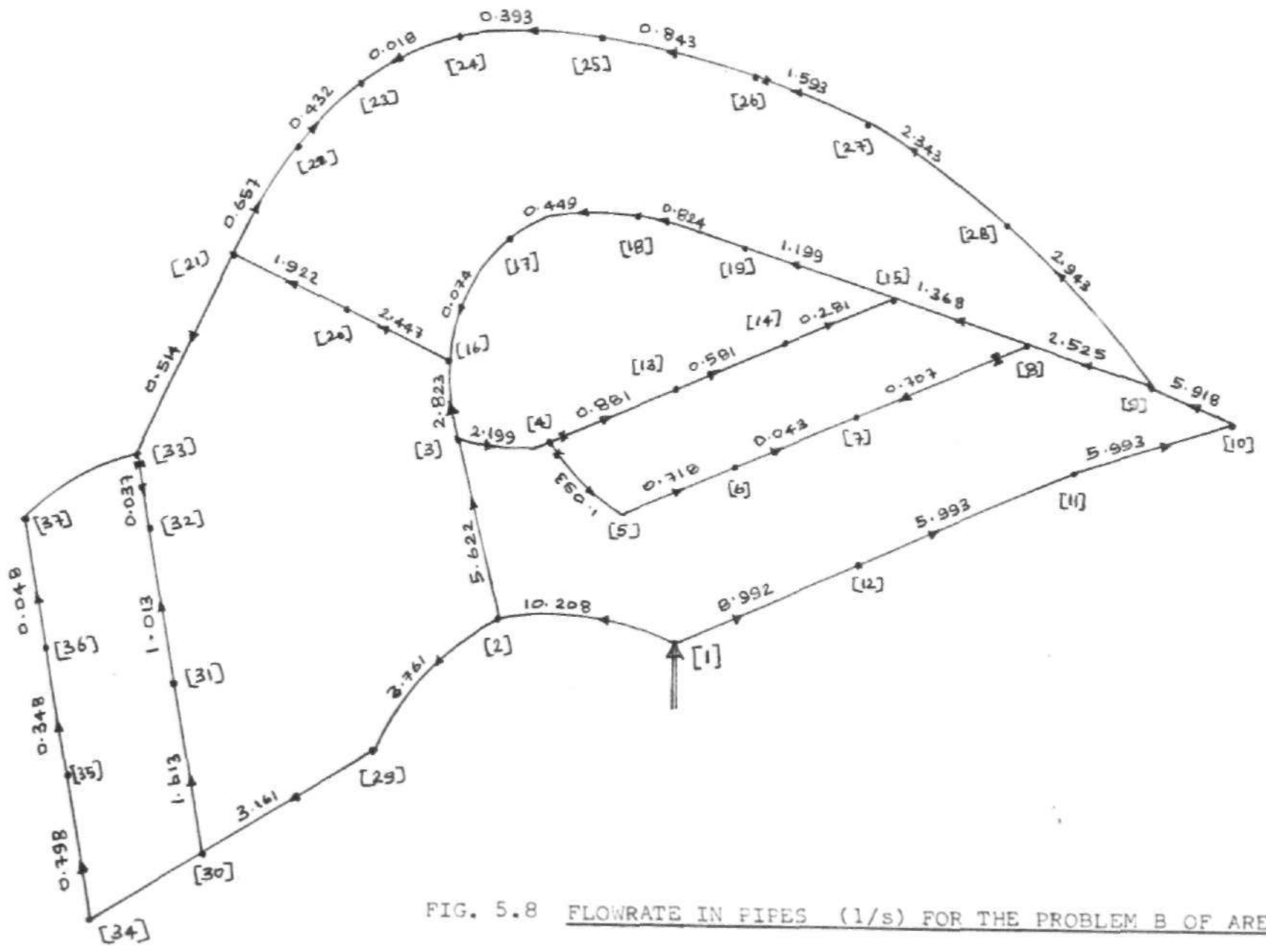


FIG. 5.8 FLOWRATE IN PIPES (l/s) FOR THE PROBLEM B OF AREA C

5.6 Residential area 'E'

5.6.1 Formulation of problems:

Here, hydraulic computations were carried out inorder to

(A) estimate the required operational pressure heads at each feeding point and to determine the one with the highest satisfaction factor:

(B) to determine the influence of changes in roughness on distribution of operational pressure heads.

5.6.2 Common data:

Pipe numbers, junction numbers and loops numbers are shown in Fig. 5.9.

NP = 18 (Number of pipes)

NJ = 15 (Number of junctions)

NL = 04 (Number of loops)

Total number of houses = 82

Consumption per house = 0.15 l/s (2.0 gpm)

Assumed water temperature = 20°C

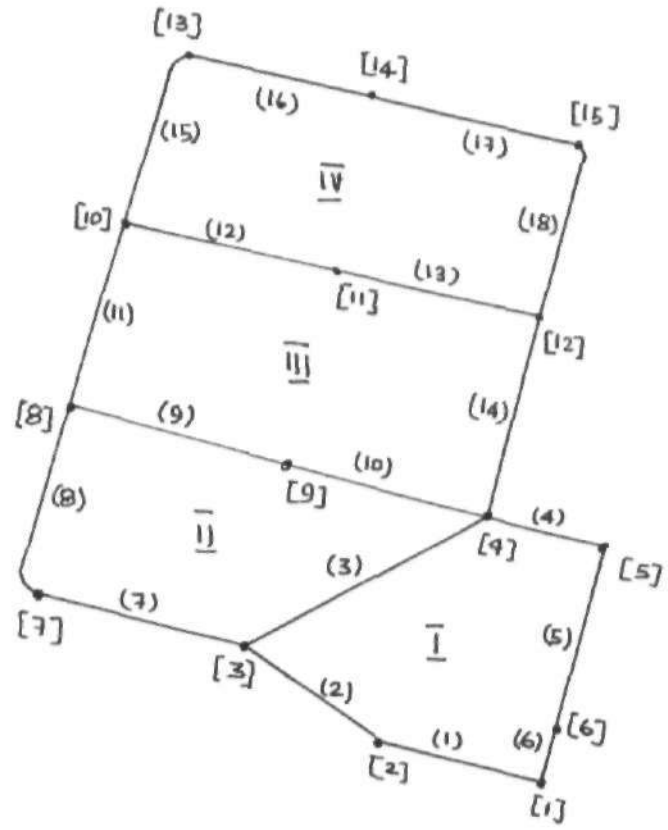
in pipe line

Lengths and diameter of pipes and also external flows are shown in Fig. 5.10.

5.6.3 Solution of problem A:

5.6.3.1 Data for hydraulic computation:

Computations were carried out seperately with three feeding points. One of the feeding point was chosen as a closest point from the central over head tank while the other two are existing supply points, namely from existing over head tank and directly from pumping main.



[ ] : Node No.

( ) : Pipe No.

I, II, ... : Loop No.

FIG. 5.9 PIPE NETWORK OF RESIDENTIAL AREA E

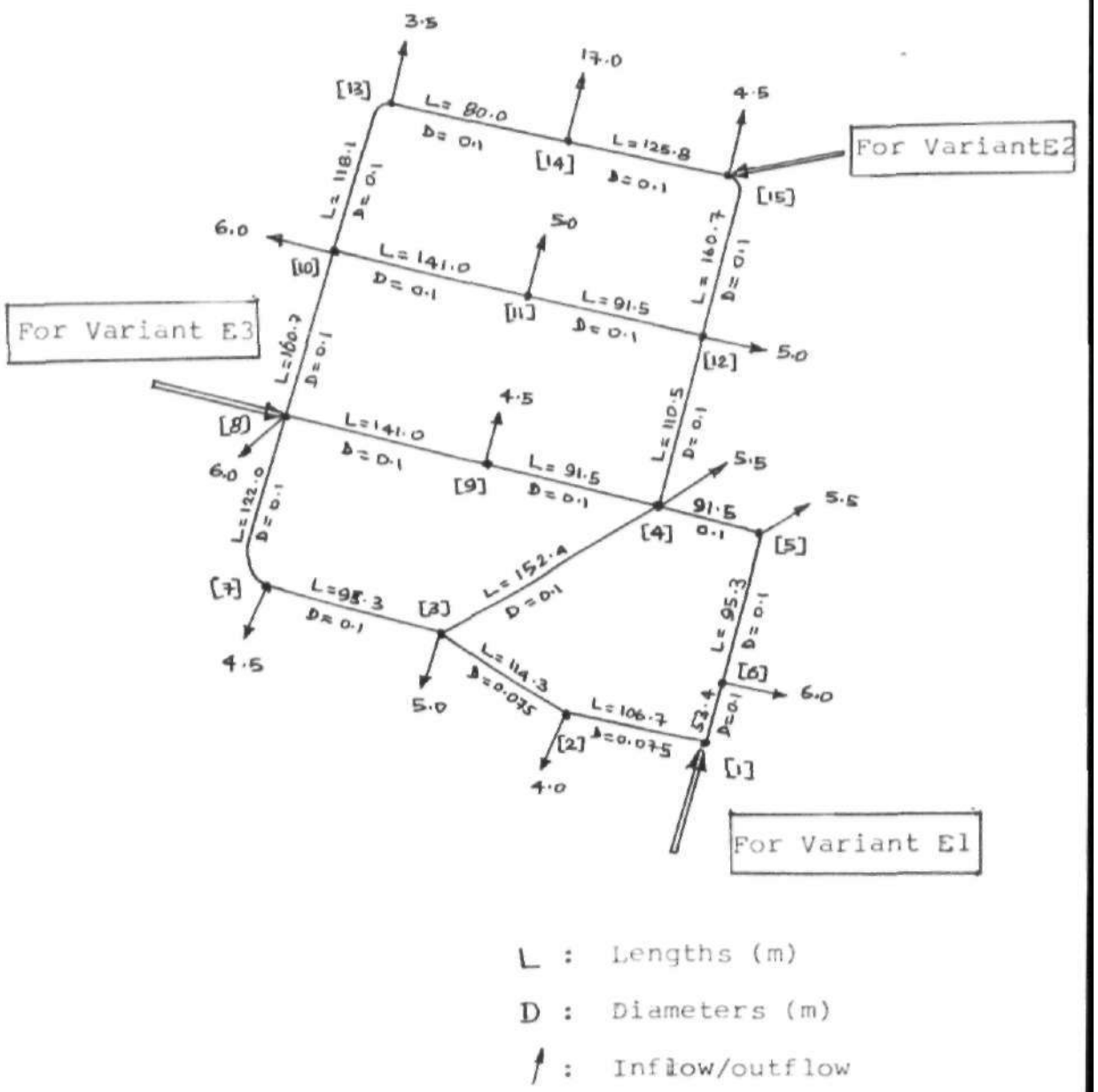


FIG. 5.10 DATA FOR AREA E PIPE NETWORK

The variants are as follows:

Variant E1: Network with a feeding point directly from pumping main:

Node number of this feeding point is 1.

External supply at the feeding point is equivalent to 82 houses.

Variant E2: Network with a feeding point closest to central over head tank:

Node number of the feeding point is 15

Demand at feeding point equivalent to 4.5 houses

Hence Net external flow at the supply Node is equivalent to (82 - 4.5)= 77.5 houses.

Variant E3: Network with a feeding point closet to over head tank:

Node number of the feeding point = 8

Demand at the feeding point equivalent to 6 houses.

Hence, Net external flow at feeding point is equivalent to (82 - 6) = 76 houses.

5.6.3.2 Results and analysis:

The flowrates in pips for all three variants obtained from computer are as shown in Figs. 5.11, 5.12 and 5.13

Operational pressure heads at feeding points:

In the first step (Eq.4.3) the relative pressure head H_r^n were computed for all three variants, are shown in table 5.7.

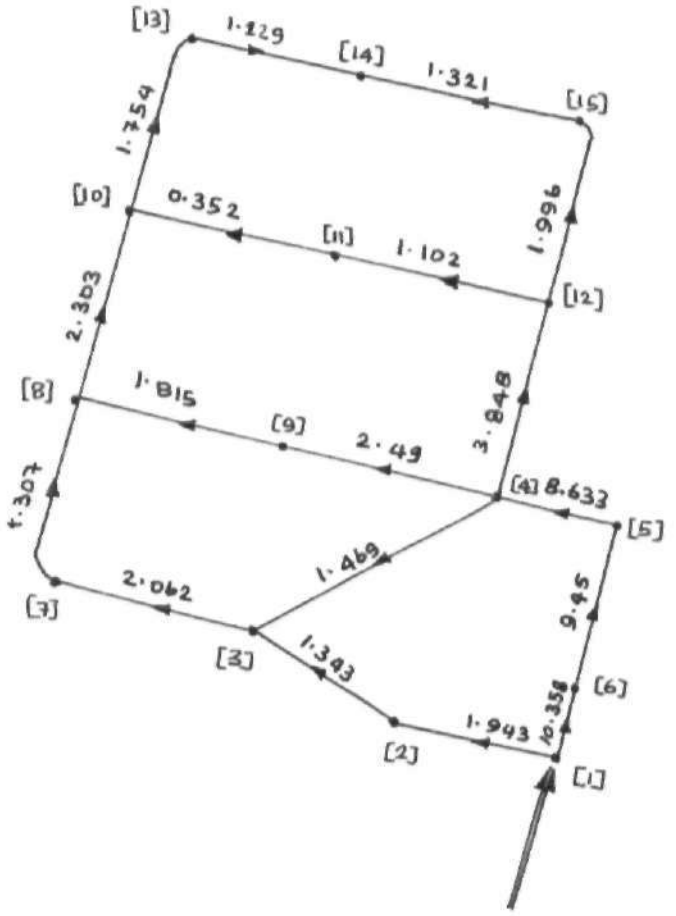


FIG. 5.11 FLOWRATE IN PIPES (l/s) FOR VARIANT E1

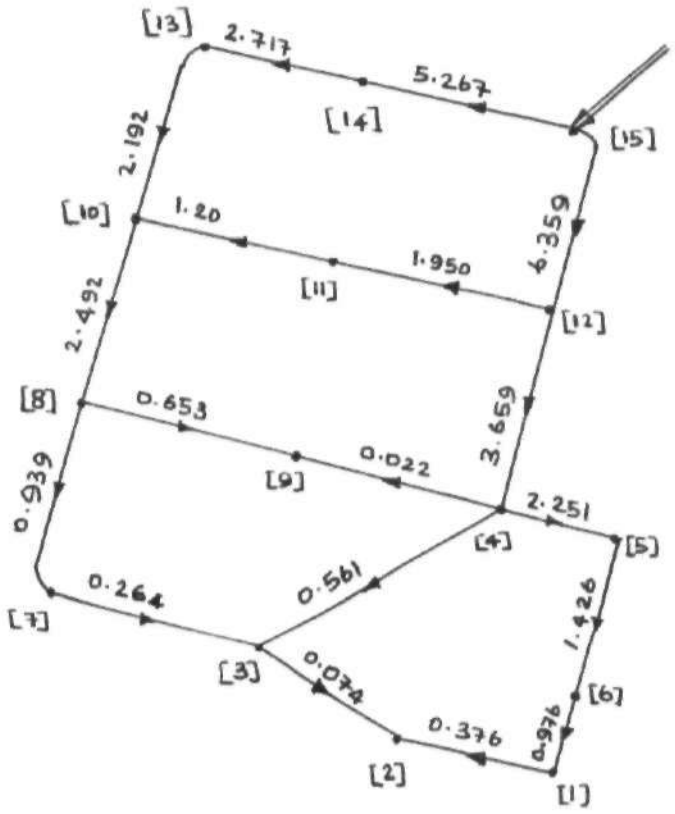


FIG. 5.12 FLOWRATE IN PIPES (1/s) FOR VARIANT E2

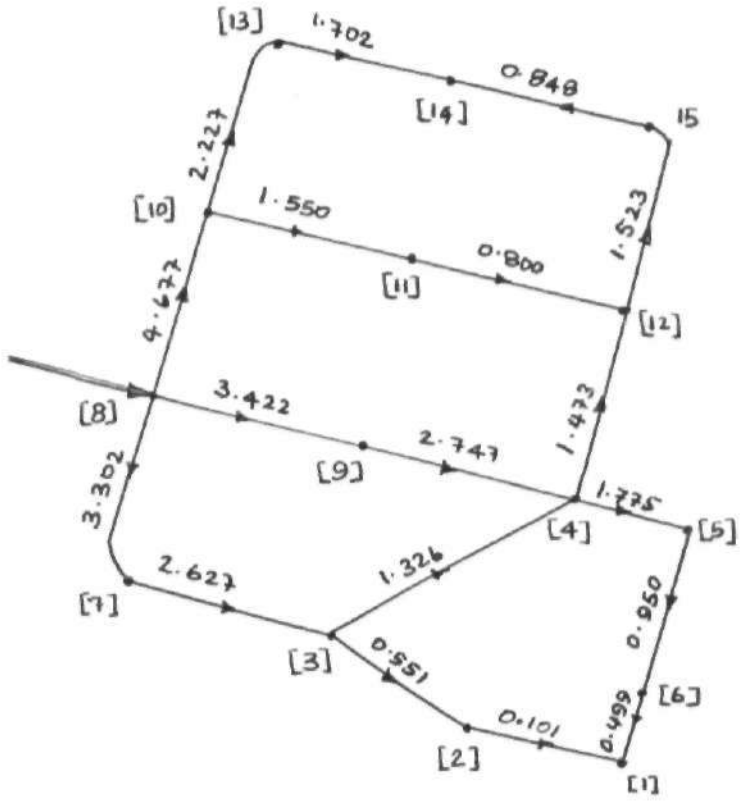


FIG. 5.13 FLOWRATE IN PIPES (l/s) FOR VARIANT E3

TABLE 5.7: RELATIVE PRESSURE HEAD AT NODES IN PROBLEM A OF AREA E

Junc. No.	Pipe No.	Topo Level	H.G.L. elevation when the pressure head at feeding point is zero H_e^n			Friction head loss in pipe			Relative pressure heads H_r^n (m)		
			E1	E2	E3	E1	E 2	E3	E1	E2	E3
1	2	3	4	5	6	7	8	9	10	11	12
01		689.131	689.131	692.843	692.5006				0.00	3.712	3.370
	01					0.201	0.013	0.023			
02		689.207	688.910	692.830	692.524				-0.297	3.623	3.317
	02					0.124	0.019	0.026			
03		691.189	688.786	692.048	692.550				-2.403	1.659	1.361
	03					0.049	0.009	0.041			
04		691.463	688.835	692.857	692.509				-2.628	1.394	1.046
	04					0.097	0.009	0.006			
05		690.808	688.933	692.848	692.503				-1.875	2.04	1.695
	05					0.119	0.004	0.002			
06		689.634	689.052	692.844	692.501				-0.582	3.21	2.867
	-					-	-	-			

1	2	3	4	5	6	7	8	9	10	11	12
07		692.546	688.730	692.950	692.635				-3.816	0.304	0.089
	08					0.036	0.018	0.164			
08		692.799	688.694	692.868	692.799				-4.105	0.069	0.000
	09					0.066	0.011	0.201			
09		691.463	688.761	692.857	692.598				-2.702	1.394	1.135
	-					-	-	-			
10		693.265	688.619	692.955	692.533				-4.646	-0.67	-1.092
	12					0.004	0.032	0.050			
11		692.683	688.623	692.987	692.483				-4.06	0.304	-0.2
	13					0.018	0.048	0.010			
12		692.439	688.641	693.035	692.473				-3.798	0.596	0.034
	-					-	-	-			
13		694.433	688.567	693.031	692.454				-5.866	-1.402	-1.979
	16					0.019	0.076	0.033			
14		693.729	688.548	693.107	692.421				-5.181	-0.621	-1.308
	17					0.034	0.387	0.016			
15		693.494	688.582	693.494	692.437				-4.912	0.000	-1.057

In every system of feeding the minimum relative pressure head $\left[H_r^n\right]_{min}$ were observed only at node No. 13, are as follows:

For Variant E1 $H_r^{13} = -5.866$m

Variant E2 $H_r^{13} = -1.402$m

Variant E3 $H_r^{13} = -1.979$m

Therefore the operational pressure head to be maintained at each feeding point are as follows from Eq. (4.4).

for Variant E1 $H_{opr}^{1} = 6.00 + 5.866 = 11.866$m

Variant E2 $H_{opr}^{8} = 6.00 + 1.402 = 7.402$m

Variant E3 $H_{opr}^{15} = 6.00 + 1.979 = 7.979$m.

Measure of satisfaction:

Operational pressure heads were computed from Eq. (4.5) and are tabulated as shown in Table 5.8.

From Eq. (5.1) percentage of satisfactions were calculated and are also tabulated in the same Table 5.8.

The computed average percentage of satisfactions using Eq. (4.7) for all three variants are as follows:

For Variant E1 = 45.7%

Variant E2 = 40.7%

Variant E3 = 43.3%

TABLE 5.8: OPERATIONAL PRESSURE HEADS AND PERCENTAGE OF SATISFACTION AT NODES IN PROBLEM **A** OF ANEX E J.I.S NETWORK

JN No.	Operational pressure heads H^n_{opr}			Diff. in pressure head = $H^n_{opr} - n_{min}$ (m)			M. O. S. percentage		
	E1	E2	E3	E1	E2	E3	E1	E2	E3
1	2	3	4	5	6	7	8	9	10
01	11.866	11.114	11.349	5.866	5.114	5.349	97.8	85.2	89.2
02	11.569	11.025	11.296	5.569	5.029	5.296	92.8	83.8	88.3
03	9.463	9.061	9.340	3.463	3.061	3.340	57.7	51.0	55.7
04	9.238	8.796	9.025	3.238	2.796	3.025	54.0	46.6	50.4
05	9.991	9.442	9.674	3.991	3.442	3.674	66.5	57.4	61.2
06	11.284	10.612	10.846	5.284	4.612	4.846	88.1	76.9	80.8
07	8.050	7.706	8.068	2.050	1.706	2.068	34.2	28.4	34.5
08	7.761	7.471	7.979	1.761	1.471	1.979	29.4	24.5	33.0
09	9.164	8.796	9.114	3.164	2.796	3.114	52.7	46.6	51.9
10	7.220	6.732	6.887	1.220	0.732	0.887	20.3	12.2	14.8

1	2	3	4	5	6	7	8	9	10
11	7.806	7.706	7.779	1.806	1.706	1.779	30.1	28.4	29.65
12.	8.068	7.998	8.013	2.068	1.998	2.013	34.5	33.3	33.6
13	6.000	6.000	6.000	0.000	0.000	0.000	0.0	0.0	0.0
14	6.685	6.780	6.671	0.685	0.780	0.671	11.4	13.0	11.2
15	6.954	7.402	6.922	0.954	1.402	0.922	15.9	23.4	15.4
				The average percentage of satisfaction		=	45.7	40.7	43.3

Design of supply lines for all three variants:

Total No. of houses = 82

Assumed demand/house = 0.15 l/s

Assumed diameter of supply line = 0.2m

For Variant S1 the lenght of supply line (L) = 640m

Variant S2 " = 195m

Variant S3 = 945m

Hence, the required rate of supply at feeding points for all three variants = 82 x 0.15 l/s

$= 12.3 \times 10^{-3} m^3/s$

Hence, the Reynold number Re

Eq. (4.8)

$$Re = \frac{4}{\pi D \nu} \cdot Q$$

$$= \frac{4 \times 12.3 \times 10^{-3}}{\times 0.2 \times 1.007 \times 10^{-6}} \frac{m^3/s}{m^3/s}$$

$$= 7.8 \times 10^4$$

The relative roughness Eq. (4.9)

$$\frac{E}{D} = 0.0000125$$

Hence, from Moody diagram the friction factor f = 0.0183

Hence, the frictional head losses h_f

Eq. (4.10)

$$h_f = \frac{8 \times 0.0183 \times (12.5 \times 10^{-3})^2}{\pi^2 \times 9.8 \times (0.2)^5} L$$

$$h_f = 0.00074 \times L$$

Hence, for variant E1 h_f = 0.474m

variant E2 h_f = 0.144m

variant E3 h_f = 0.700m

As the calculated frictional head losses in supply lines are so small to accept, the assumed diameter of pipe 0.2m is taken as the design value for the supply lines.

Computation of the tank height:

(a) variant E1:

The minimum operational pressure head required at feeding point = 11.866m

Topo level at feeding point = 689.131m

Hence, H.G.L. elevation at feeding point = 689.131 + 11.866

= 700.997m

Frictional head losses on supply line from central over head tank to feeding point = 0.474m

Hence, H.G.L. elevation required at the bottom of the tank = 700.997 + 0.474 = 701.471m

Topo level at bottom of the tank = 698.171m

Hence, pressure head required at bottom of the tank

= 701.471 - 698.171 = 3.30m

(b) Variant E2:

At feeding point:

Minimum operational pressure head required = 7.402m

Topo level = 693.494m

Hence, H.G.L. elevation required = 693.494 + 7.402

= 700.896m

Frictional head losses in supply line from central over head tank to feeding point = 0.144m

At bottom of the tank:

H.G.L. elevation required = 700.896. + 0.144

= 701.040m

Topo level = 698.171m

Hence, pressure head required = 701.040 - 698.171

= 2.869m.

(c) Variant E3

At feeding point:

Minimum pressure head required = 7.979m

Topo level = 690.854m

Hence, H.G.L. elevation required = 690.854 + 7.979

= 700.778m

At bottom of the tank:

H.G.L. elevation required = 700.778 + 0.700

= 701.478m

Topo level = 698.171m

Hence, pressure head required = 701.478 - 698.171

= 3.307m

Table 5.9 Comparison of tank heights, percentage of satisfactions and required operational pressure head at feeding points for all three variants in problem A of Area E pipe network:

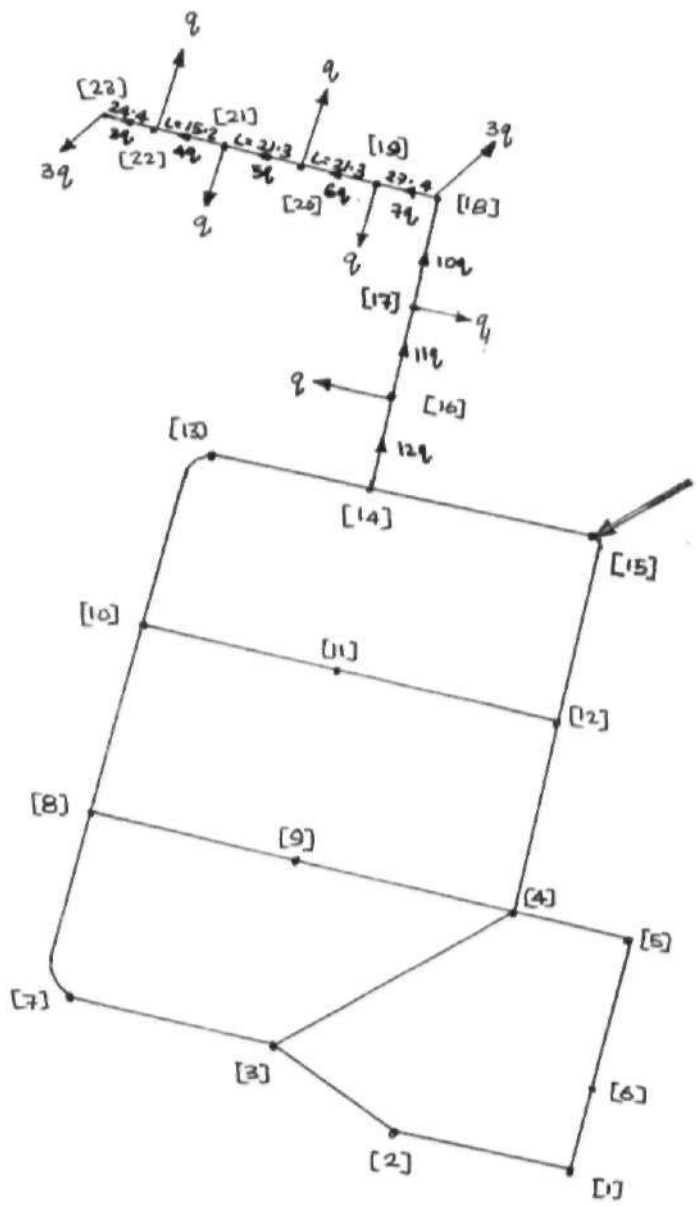
	Variant E1	Variant E2	Variant E3
Height of Tank (m)	3.30	2.87	3.31
Average percentage of satisfaction	45.7	40.7	43.3
Operational pressure head at feeding point (m)	11.87	7.40	7.98

Check on satisfactory operational pressure heads for houses from No. 1 to No. 12:

The supply lines to the above houses were not included in the Fig. 5.10 which showing the pipe network of residential area E. It is because, only the close loops were considered in linear theory method for analysis of pipe network.

The supply line to these houses is being tapped from node 14.

Fig. 5.14 shows the flowrates, lengths between the nodes and the discharge from the nodes (in equivalent houses) for the houses from No. 1 to No. 12.



[ ] : Node No.

L : Pipe Length (m)

q : Consumption Rate/House (l/s)

FIG. 5.14 DIAGRAM SHOWING THE SUPPLY LINE FOR THE HOUSES FROM NO. 1 TO 12 FROM NODE NO. 14 OF THE PIPE NETWORK DIAGRAM OF VARIANT E2

Diameter of supply main = 0.075m

Consumption rate per house q = 0.15 l/s

The reynold number:

Eq. (4.8)

$$Re = \frac{4}{\pi D \nu} \cdot Q = 1.686 \times 10^7 \, q$$

Relative roughness:

$$\frac{E}{D} = 0.0000333$$

The frictional head losses in pipes h_f:

Eq. (4.10),

$$h_f = 3.49 \times 10^4 \, fLq^2$$

The computed results are as tabulated in table 5.10.

Table 5.10 Friction head losses in pipes supplying water to house No. 1 to 12 in area E

Node Nos. for the pipe	Reynold No. Re x 10^4	Friction factor f	Friction head losses h_f (m)
14 - 16	3.03	0.0235	0.132
16 - 17	2.70	0.0230	0.073
17 - 18	2.53	0.0232	0.809
18 - 19	1.77	0.0237	0.025
19 - 20	1.51	0.0255	0.015
20 - 21	1.20	0.0270	0.011
21 - 22	1.01	0.0290	0.006
22 - 23	0.76	0.031	0.005

From the table above, the total friction head losses in pipes between node 14 to node 23. = 0.356m.

But operational pressure head at node 14 as in variant E3. = 6.671m

Hence, operational pressure head at the last node No. 23 = 6.671 - 0.356

= 6.315m

Hence, the operation pressure head at the last node is more than the minimum pressure head requirement, the calculated operational pressure heads at the feeding point could satisfy the minimum pressure head requirements at all nodes which supply water to all the above houses from E_1 - E_{12}.

Comparison of the system of feedings:

For all three variants, elevation of water surfaces in the tanks are chosen as the same elevation of water surface in the tank of the variant E3, because the variant E3 has the highest water surface which could satisfy the minimum standard pressure head requirement at all nodes for all three variants.

Hence, from the previous subsection 5.6.3.2. The chosen water surface elevation in the tank = 3.31m.

The H.G.L. elevation at the bottom of the tank

= 701.478m.

Operational pressure heads at feeding point as the result of assumed water surface level in the tank:

For Variant E2:

H.G.L. elevation at bottom of the tank

= 701.478m.

The friction head losses in supply line between the common tank and feeding point = 0.14m.

Hence, H.G.L. elevation at feeding point = 701.478 - 0.144

= 701.33m

Hence, increased operational pressure head at feeding point = 701.334 - 693.494

= 7.840m.

Variant E1

H.G.L. elevation at bottom of the tank = 701.478m

the friction head losses on supply line between the common tank and feeding point = 0.474m.

Hence, H.G.L. elevation at feeding point = 701.478 - 0.474

= 701.004m

but topo level at feeding point = 689.131m

Hence, increased operational pressured at feeding point = 701.004 - 689.131

= 11.873m

Computation of percentage of satisfactions:

Variant S2

From the previous result the increased operational pressure head at feeding point as the results of assumed water surface level in the tank = 7.840m.

From page number 77 , the operational pressure head at feeding point to satisfy the minimum standard pressure heads at nodes in the network = 7.402m.

Hence, increase in operational pressure head at feeding point computed as the difference of above .

= 7.840 - 7.402

= 0.438m

Hence, the increase in operational pressure heads at all nodes due to assumed water surface level in the tank is constant, equals to 0.438m.

Hence, average increase in percentage of satisfaction from Eq. (4.7) is as follow:

$$= \frac{1}{N} \sum \frac{H^n_{opr}}{h_{min}} \times 100$$

$$= \frac{H^n_{opr}}{h_{min}} \times 100$$

$$= \frac{2.438}{6.0} \times 100$$

$$= 7.3\%$$

Hence, average percentage of satisfaction = 7.3 + 40.7

= 48.0

Variant E1

New operational pressure head at feeding point

= 11.873m

Hence, increase in operational pressure head at feeding point = 11.873 - 11.866

= 0.007m

Hence, there will be an increase of 0.007m operational pressure head at all nodes.

Hence, average increase in percentage of satisfaction

$$= \frac{0.007}{5.0} \times 100$$

$$= 0.12$$

Hence, the new percentage of satisfaction = 45.7 + 0.12

= 45.82

Now the length of supply mains were measured and compared for economy.

Table 5.11 Comparison of lengths of supply lines and percentage of satisfactions:

	Variant E1	Variant E2	Variant E3
Length of the supply line (m)	640	195	945
Percentage of satisfaction	45.8	48.0	43.3

5.6.3.3 Conclusions:

The best recommended feeding point for the residential area 3 is the variant E2 which is the closest node from the central over head tank.

5.6.4 Solution of problem B

5.6.4.1 Data for hydraulic computation:

All the data for the problem B are same as for the problem A of the previous subsection 5.2.3 except the roughness of the pipes.

Assumed roughness of the Asbestos cement pipes in this section is 0.0005 in (0.0125mm) instead 0.0001 inch in all other sections as estimated in subsection 4.7.

5.6.4.2 Results and analysis:

The flow rates and frictional head losses in pipes obtained from computer analysis are tabulated in Table No. 5.12.

From the above table, it is observed that the changes in frictional head losses in the pipes are very very small while flow-rates are same.

Error in operational pressure head at nodes:

The error in operational pressure head at nodes due to changes in roughness of the pipes are computed as follows:

$$\text{Error} = \frac{H^n_{opr\ El} - H^n_{opr\ B}}{H^n_{opr\ El}} \quad .. \quad .. \quad (5.2)$$

where $H^n_{opr\ El}$ = Operational pressure head at node 'n' in variant El of the problem A.

$H^n_{opr\ B}$ = Operational pressure head at node 'n' computed as in problem B.

Table 5.12 Compa rison of florates and friction head losses in pipe when chaning its roughness:

NP	Flowrates (1/s)		Friction nead losses (m)	
	Problem A Variant E1	Problem B	Problem A Variant E2	Problem B
01	1.94	1.94	0.221	0.224
02	1.34	1.34	0.124	0.125
03	1.47	1.47	0.049	0.050
04	8.63	8.63	0.097	0.093
05	9.46	9.46	0.119	0.121
06	10.36	10.36	0.079	0.080
07	2.06	2.06	0.056	0.056
08	1.39	1.39	0.036	0.036
09	1.82	1.82	0.066	0.065
10	2.49	2.49	0.075	0.075
11	2.30	2.30	0.076	0.077
12	0.35	0.35	0.004	0.004
13	1.10	1.10	0.018	0.016
14	3.85	3.84	0.194	0.197
15	1.75	1.76	0.052	0.052
16	1.23	1.23	0.019	0.019
17	1.32	1.32	0.034	0.034
18	2.00	2.00	0.059	0.059

The computed results are tabulated in the table No. 5.13.

5.6.4.3 Conclutions:

The maximum error in operational pressure head at nodes due to changes in roughness of the pipe is so small as 0.1% that could be negleted.

TABLE 5.13: PERCENTAGE OF ERROR IN OPERATIONAL PRESSURE HEAD DUE TO CHANGES IN ROUGHNESS OF THE PIPE

JN 1	Pipe No. 2	Topo level 3	Friction head losses 4	H.G.L. elevation when feeding pressure head is zero (m) 5	Relative pressure head H_r^n (m) 6	Operational pressure head H_{opr}^n 7 B	Operational pressure head H_{opr}^n 8 A	Percentage of error 9
01		689.131		689.13	-	11.872	11.866	0.05
	01		0.224					
02		689.207		688.907	-0.30	11.572	11.569	0.03
	02		0.125					
03		691.189		688.782	-2.407	9.465	9.463	0.02
	03		0.050					
04		691.463		688.832	-2.631	9.241	9.238	0.03
	03		0.099					
05		690.808		688.931	-1.877	10.001	9.991	0.10
	05		0.121					
06		689.634		689.052	-0.582	11.290	11.284	0.05

1	2	3	4	5	6	7	8	9
07		692.546		688.726	-3.820	8.052	8.050	0.03
	08		0.036					
08		692.799		688.690	-4.109	7.763	7.761	0.03
	09		0.065					
09		691.463		688.755	-2.708	9.164	9.164	0.00
	-		-					
10		693.265		688.613	-4.652	7.220	7.220	0.00
	12		0.004					
11		692.683		688.617	-4.065	7.807	7.806	0.01
	13		0.018					
12		692.439		688.635	-3.804	8.068	8.068	0.00
	-		-					
13		694.433		688.561	-5.872	6.000	6.000	0.00
	16		0.019					
14		693.729		688.542	-5.187	6.685	6.685	0.00
	17		0.034					
15		693.494		688.576	-4.918	6.954	6.954	0.00

5.7 Residential area 'F'

5.7.1 Formulation of problems:

Here, hydraulic computations were carried out inorder to

(A) estimate the required operational pressure heads at feeding point

(B) to determine the influences of changes in rate of consumption on the distribution of pressure heads

(C) to determine the influence of simplification of the sheme of network on distribution of pressure heads.

5.7.2 Common data:

Total number of houses = 160.

5.7.3 Solution of problem A

5.7.3.1 Data for hydraulic computations:

Computations were carried out seperately for all three schemes of network.

Variant F1:

The scheme of network with pipe No., junction No., and loop No. are shown in Fig. 5.15

NP = 30 (No. of pipes)

NJ = 26 (No. of junctions)

NL = 05 (No. of loops)

External Inflow at feeding point is equivalent to 160 houses. Pipe lengths and diameters, flowrates at nodes are shown in Fig. 5.16.

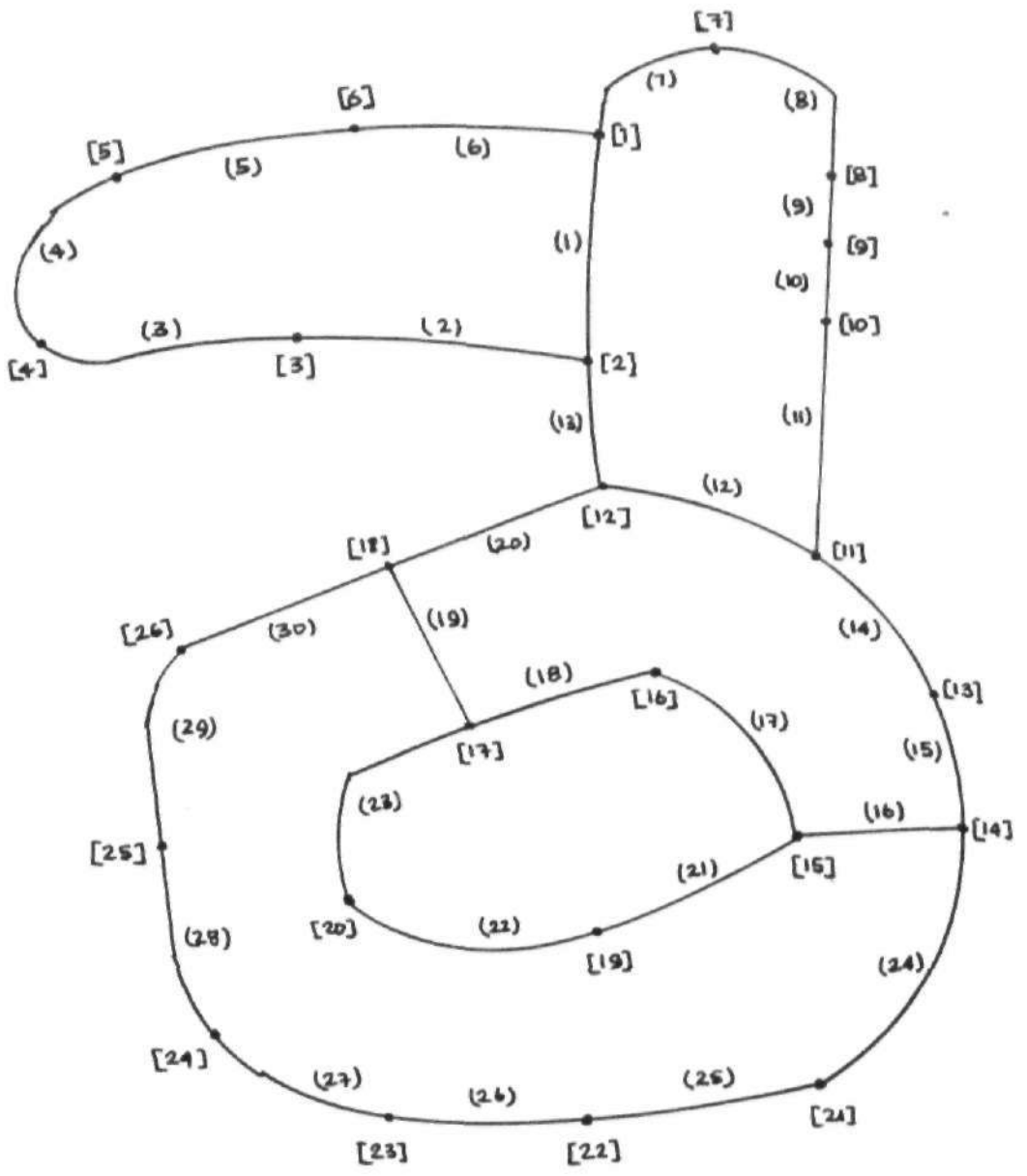


FIG. 5.15 PIPE NETWORK DIAGRAM FOR VARIANT F1

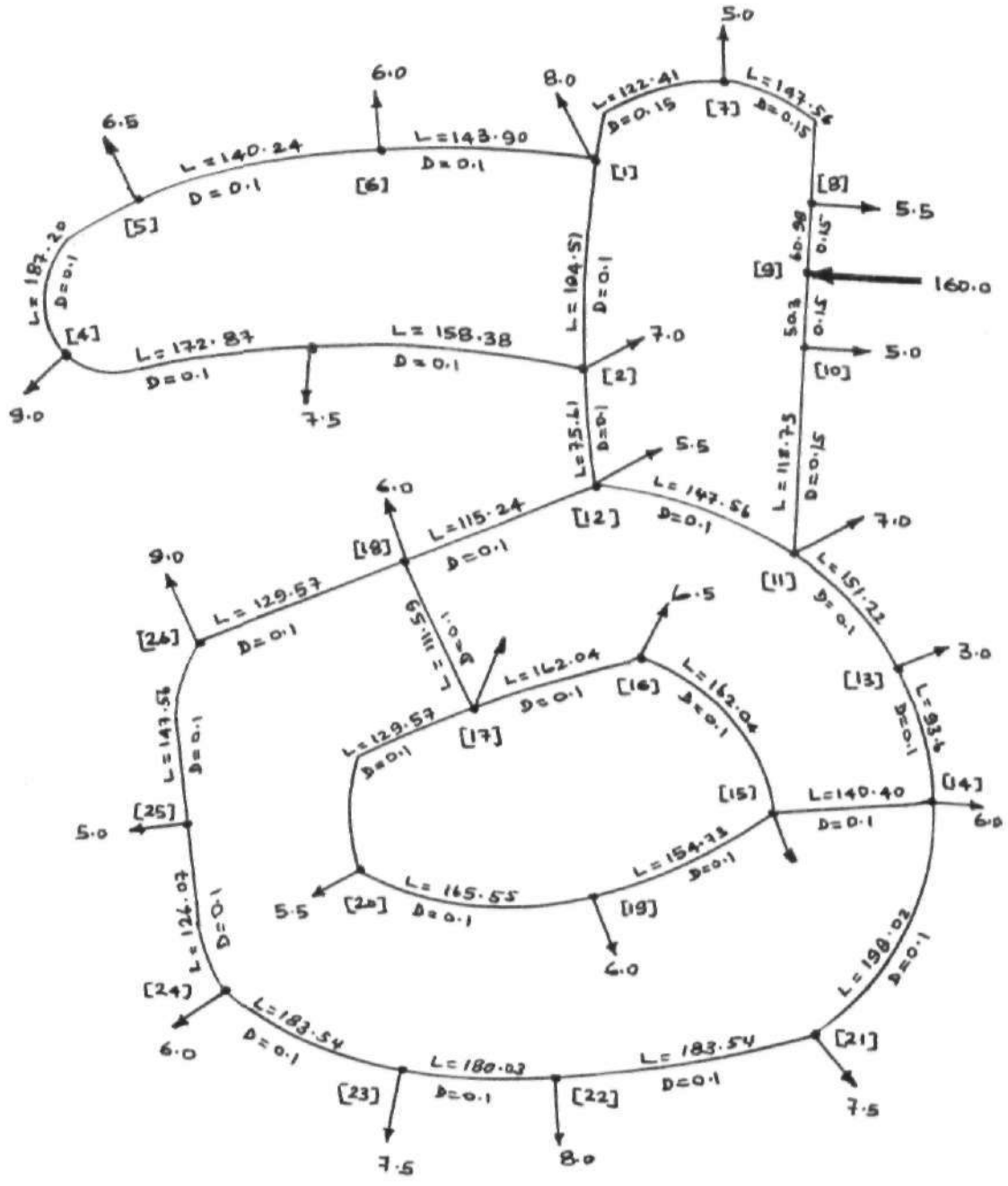


FIG. 5.16 DATA FOR VARIANT F1

Variant F2: (simplified network of variant F1)

The scheme of network with pipe No. junction No. and loop No. are shown in Fig. 5.17.

NP = 27 (No. of pipes)

NJ = 24 (No. of junctions)

NL = 04 (No. of loops)

External inflow at feeding point, is equivalent to 160 houses. Lengths and diameter of pipes, flowrates at nodes are shown in Fig. 5.18.

Variant F3: (simplified network of variant F2)

The scheme of the pipe network with pipe No., junction No., and loop No. are shown in Fig. 5.19.

NP = 17 (No. of pipes)

NJ = 14 (No. of junction)

NL = 04 (No. of loops)

No. of houses at feeding point = 10.5

Hence, Net external inflow at feeding point is equivalent to (160-10.5) = 149.5 houses. Length and diameter of the pipes, No. of houses at nodes are shown in Fig. 5.20.

5.7.3.2 Results and analysis:

Frictional head losses in pipes for all three variants are tabulated in tables 5.14, 5.15 and 5.16 along with topo levels of the nodes.

Relative pressure heads:

From Eq. (4.3) the relative pressure heads at nodes respect to feeding point were computed, and are tabulated in the Table Nos. 5.14, 5.15 and 5.16.

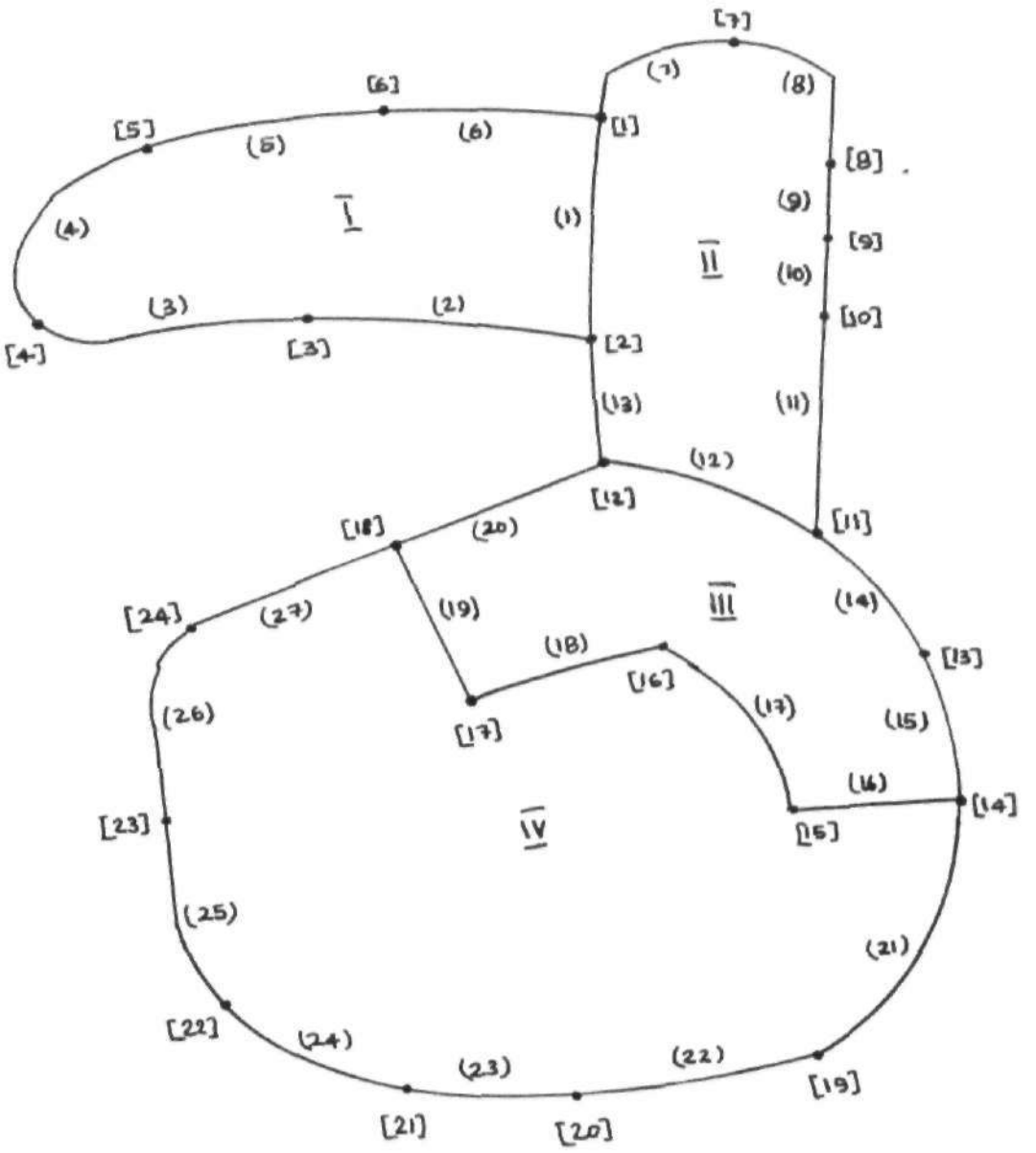


FIG. 5.17 PIPE NETWORK DIAGRAM FOR VARIANT F2

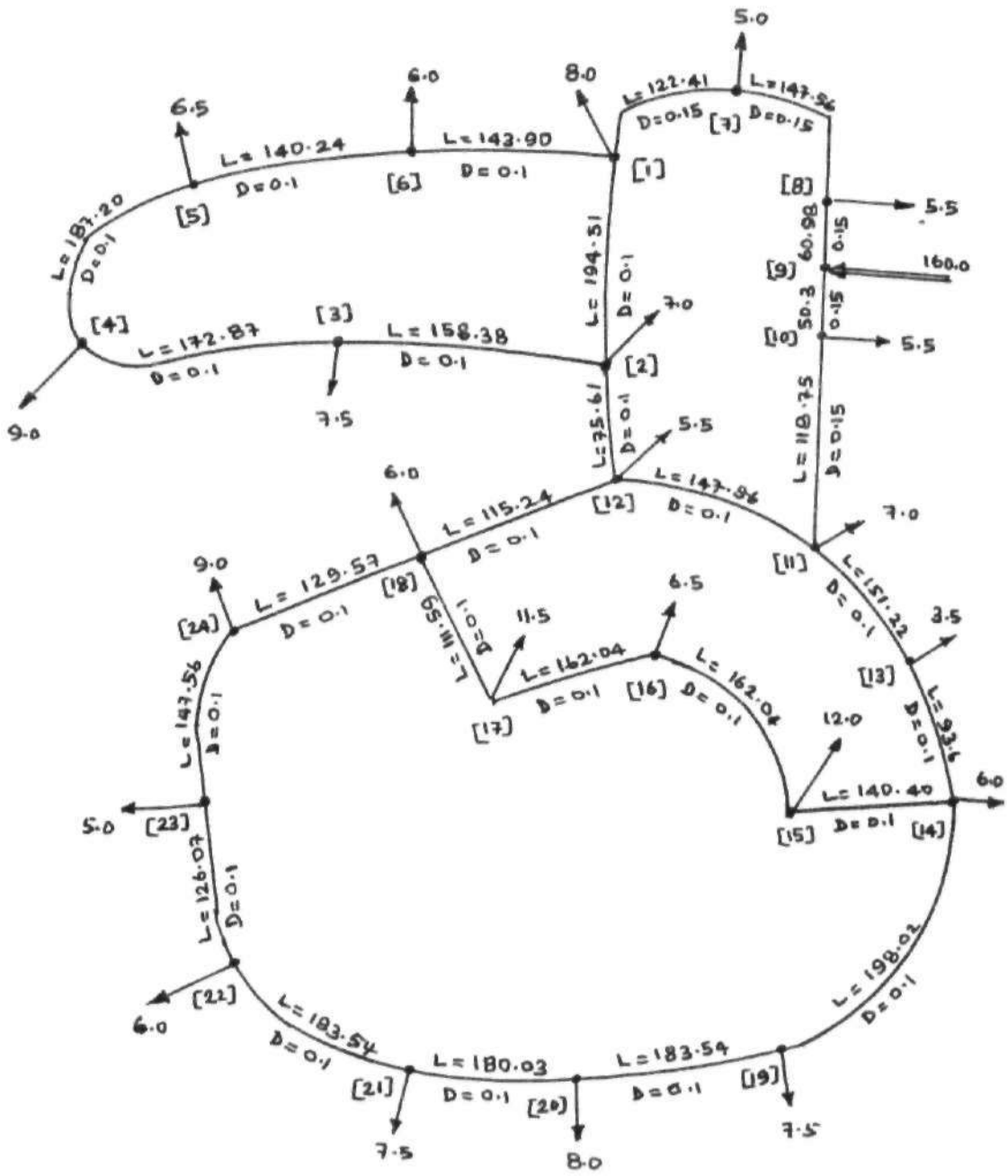
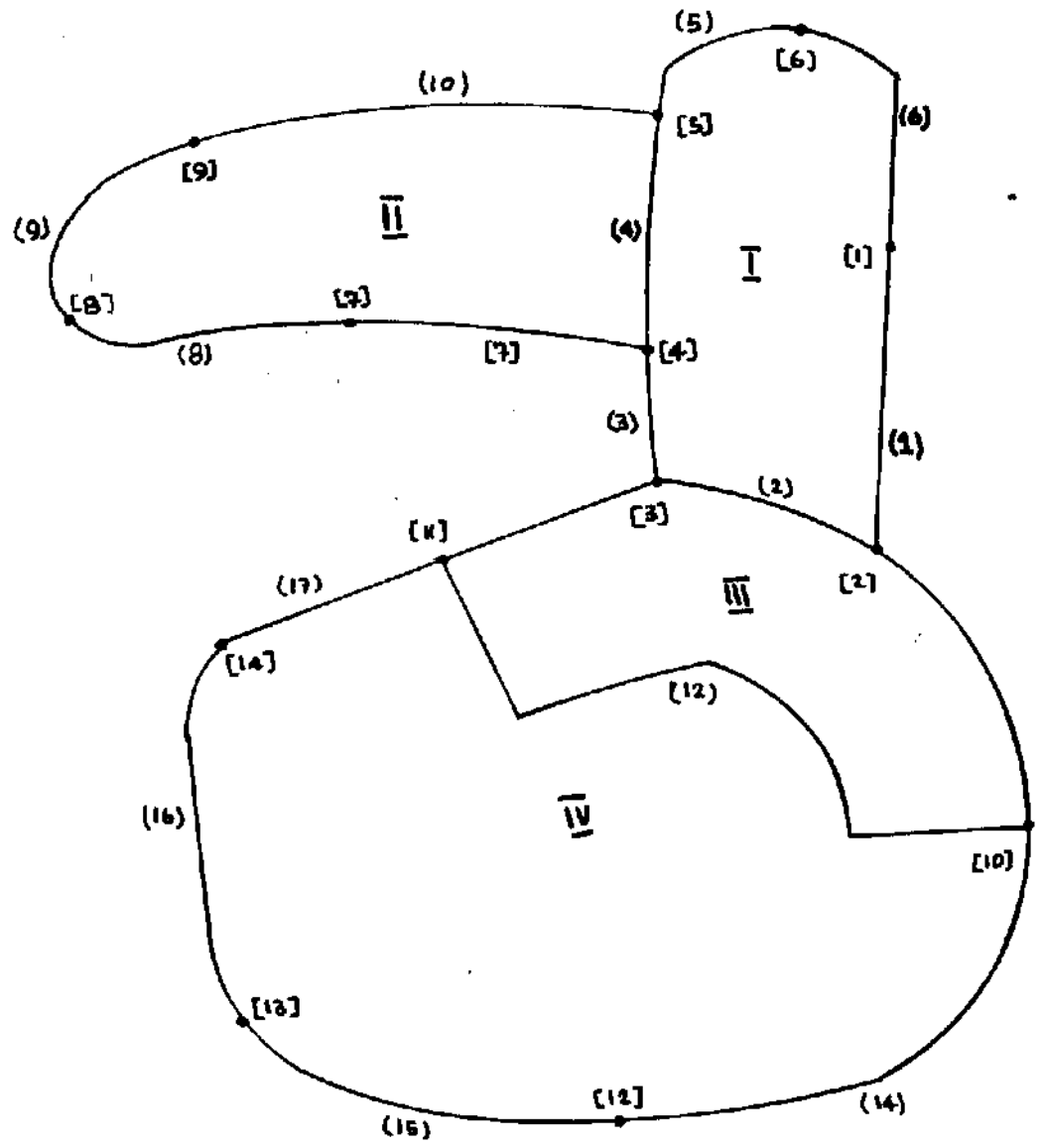


FIG. 5.18 DATA FOR VARIANT F2



209330

FIG. 5.19 PIPE NETWORK DIAGRAM FOR VARIANT F3

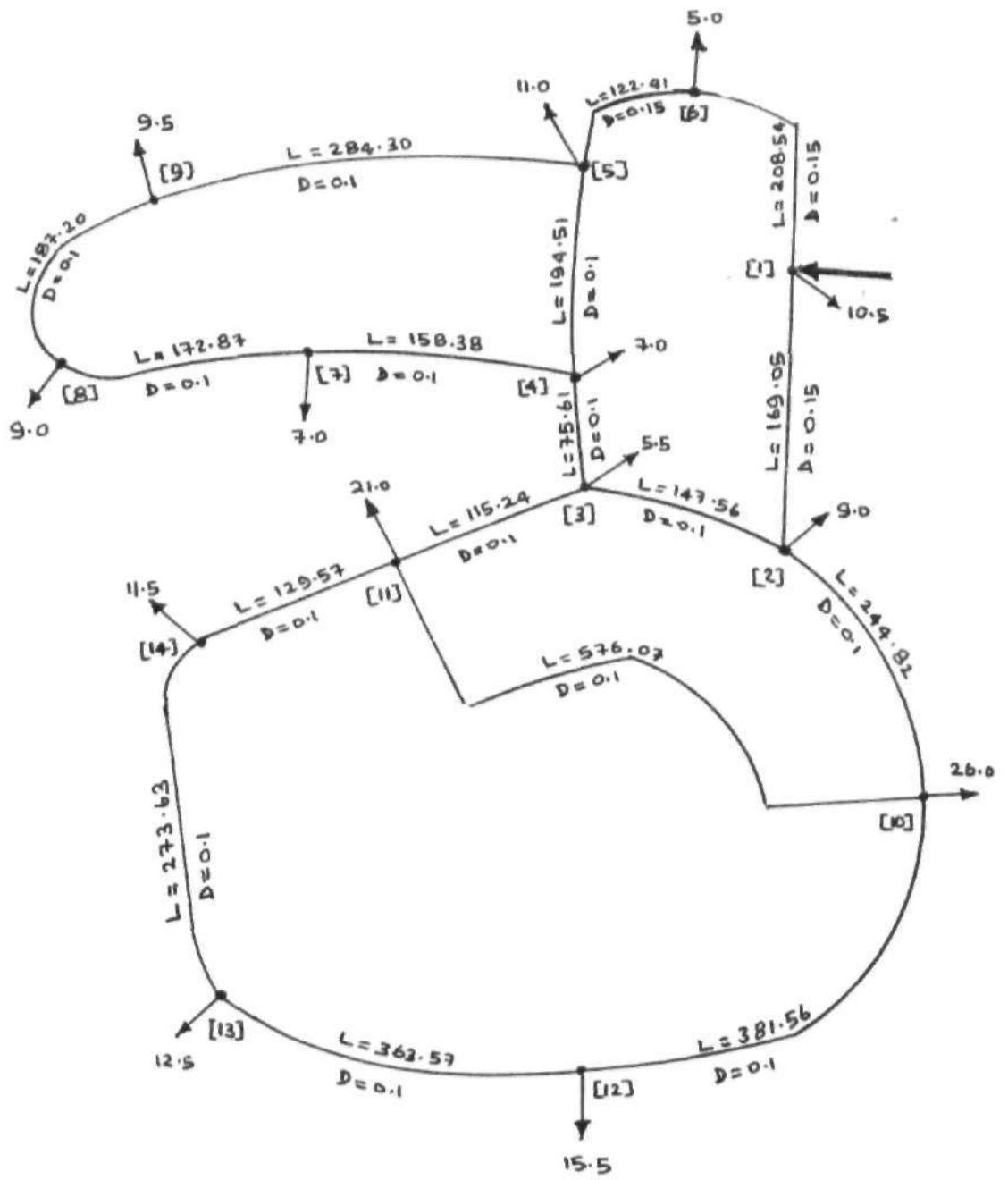


FIG. 5.20 DATA FOR VARIANT F3

TABLE 5.14: OPERATIONAL PRESSURE HEAD AT NODES FOR VARIANT F2 IN PROBLEM A OF AREA F PIPE NETWORK

JN No.	Pipe No.	Topo levels	H.G.L. elevation H_e^n (m)	Fric. head losses in pipes (m)	H_r^n (m)	H_{opr}^n
1	2	3	4	5	6	7
01		681.908	683.851		1.943	8.100
	01			0.608		
02		680.183	683.243		3.060	9.217
	02			0.009		
03		678.963	683.234		4.271	10.428
	03			0.023		
04		675.305	683.257		7.952	14.109
	04			0.127		
05		678.049	683.384		5.335	11.492
	05			0.180		
06		680.183	683.564		3.381	9.538
	-			-		
07		683.534	684.106		0.572	6.729
	08			0.341		
08		684.604	684.447		-0.157	6.000
	09			0.157		
09		684.604	684.604		0.000	6.157
	10			0.068		
10		684.604	684.536		-0.068	6.089
	11			0.140		
11		683.384	684.396		1.012	7.169

1	2	3	4	5	6	7
	12			0.159		
12		680.488	684.237		3.749	9.906
	-			-		
13		683.537	683.949		0.412	6.569
	15			0.228		
14		682.774	683.721		0.947	7.104
	16			0.010		
15		681.555	683.711		2.156	8.313
	17			0.007		
16		681.402	683.718		2.316	8.473
	18			0.049		
17		678.354	683.767		5.413	11.570
	19			0.201		
18		678.354	683.968		5.614	11.771
	-			-		
19		679.268	683.702		4.434	10.591
	22			0.017		
20		677.030	683.719		6.689	12.846
	-			-		
21		679.268	683.480		4.212	10.369
	25			0.102		
22		678.354	683.378		5.024	11.181
	26			0.020		
23		676.677	683.358		6.681	12.838
	27			0.005		

1	2	3	4	5	6	7
24		674.390	683.363		8.973	15.130
	28			0.030		
25		674.695	683.393		8.698	14.355
	29			0.080		
26		675.915	683.473		7.558	13.715

TABLE 5.15: OPERATIONAL PRESSURE HEAD AT NODES FOR VARIANT F2 IN PROBLEM A OF AREA F PIPE NETWORK

JN No. 1	Pipe No. 2	Topo Levels 3	H.G.L. elevation H_e^n (m) 4	Fric. head losses in pipes (m) 5	H_r^n (m) 6	H_{opr}^n 7
01		681.908	684.045		2.137	8.256
	01			0.593		
02		680.183	683.452		3.269	9.388
	02			0.011		
03		678.963	683.441		4.478	10.597
	03			0.022		
04		675.305	683.463		8.158	14.277
	04			0.124		
05		678.049	683.587		5.538	11.657
	05			0.177		
06		680.183	683.764		3.581	9.700
	-			-		
07		683.534	684.231		0.697	6.816
	08			0.254		
08		684.604	684.485		-0.119	6.000
	09			0.119		
09		684.604	684.604		0.000	6.119
	10			0.095		
10		684.604	684.509		-0.095	6.024
	11			0.200		
11		683.384	684.309		0.925	7.044
	12			0.350		
12		680.488	683.959		3.471	9.590

1	2	3	4	5	6	7
	-			-		
13		683.537	683.799		0.262	6.381
	15			0.265		
14		682.774	683.534		0.760	6.879
	16			0.059		
15		681.555	683.475		1.920	8.039
	17			0.005		
16		681.402	683.480		2.078	8.197
	18			0.030		
17		678.354	683.510		5.156	11.295
	19			0.111		
18		678.354	683.621		5.267	11.386
	-			-		
19		679.268	683.382		4.114	10.233
	22			0.047		
20		678.354	683.335		4.981	11.100
	23			0.006		
21		676.677	683.329		6.652	12.771
	24			0.032		
22		674.390	683.361		8.971	15.090
	25			0.066		
23		674.695	683.427		8.732	14.851
	26			0.138		
24		675.915	683.565		7.650	13.769

TABLE 5.1G: OPERATIONAL PRESSURE HEAD AT NODES FOR VARIANT F3 IN PROBLEM A OF AREA F PIPE NETWORK

JN No. 1	Pipe No. 2	Topo Levels 3	H.G.L. elevation H_e^n (m) 4	Fric. head losses in pipes (m) 5	H_r^n (m) 6	H_{opr}^n 7
01		684.604	684.604		0.000	6.000
	01			0.370		
02		683.384	684.234		0.850	6.850
	02			0.461		
03		680.488	683.773		3.285	9.285
	03			0.051		
04		680.183	683.824		3.641	9.641
	04			0.381		
05		681.908	684.205		2.297	8.297
	05			0.134		
06		683.534	684.339		0.805	6.805
	-			-		
07		678.963	683.806		4.843	10.843
	08			0.003		
08		675.305	683.809		8.504	14.504
	09			0.070		
09		678.049	683.879		5.830	11.830
	-			-		
10		682.774	683.212		0.438	6.438
	12			0.013		

1	2	3	4	5	6	7
11		678.354	683.225		4.871	10.871
	-			-		
12		678.354	682.918		4.564	10.564
	15			0.012		
13		674.390	682.906		8.516	14.516
	16			0.125		
14		675.915	683.031		7.116	13.116

The observed minimum relative pressure heads are as follows:

For Variant F1 $H_r^8 = -0.157$

Variant F2 $H_r^8 = -0.119$

Variant F3 $H_r^1 = 0.000$

Operational pressure heads at feeding point:

From Eq. (4.4), the operational pressure head H_{opr}^m at feeding points are as follows:

For Variant F1 = 0.157 + 6.0 = 6.157m

Variant F2 = 0.119 + 6.00 = 6.119m

Variant F3 = 0.00 + 6.00 = 6.000m

The computed operational pressure heads at other nodes from Eq. (4.5) are tabulated in the Tables 5.14, 5.15 and 5.16.

5.7.3.3 Conclusion:

Operational pressure head required at feeding point is 6.157m to satisfy all nodes in the network.

5.7.4 Solution of problem B

Three variants with different rate of consumption as shown in subsection 5.3.4.2 were used to determine the influence on distribution of pressure heads.

5.7.4.1 Common data:

For all three variants, the pipe network diagram of variant F1 is being used.

5.7.4.2 Consumption rates for different variants:

The asumed consumption rates are tabulated in Table 5.17.

Consumption rates of all three variants in problem B of area F:

Variants	Consumption/house (gpm)	(l/s)
f1	1.7	0.1275
f2	1.3	0.0975
f3	2.7	0.2025

5.7.4.3 Results and analysis:

Frictional head losses in pipes are tabulated in Table 5.18 for all three variants.

Relative pressure heads:

From Eq. (4.3) the relative pressure heads respect to feeding point were computed and the results are tabulated in the Table No. 5.18.

The observed minimum relative pressure heads are as follows:

Variant f1 $H_r^8 = -0.117m$

Variant f2 $H_r^8 = -0.073m$

Variant f3 $H_r^8 = -0.269m$

TABLE 5.18: RELATIVE PRESSURE HEAD AT NODES OF ALL VARIANTS IN PROBLEM B OF RESIDENTIAL AREA F

Node No.	Pipe No.	Topo Levels (m)	H.G.L. elevation when feeding pressure head is zero H_e^n (m)			Friction head losses in pipes (m)			Relative pressure heads H_r^n (m)		
			f1	f2	f3	f1	f2	f3	f1	f2	f3
1	2	3	4	5	6	7	8	9	10	11	12
01		681.908	684.041	684.255	683.314				2.133	2.347	1.406
	01					0.455	0.282	1.040			
02		680.183	683.586	683.973	682.274				3.403	3.790	2.091
	02					0.008	0.006	0.012			
03		678.963	683.578	683.967	682.262				4.615	5.004	3.299
	03					0.018	0.011	0.040			
04		675.305	683.596	683.978	682.302				8.291	8.673	6.997
	04					0.096	0.060	0.216			
05		678.049	683.692	684.038	682.518				5.643	5.989	4.469
	05					0.135	0.084	0.307			
06		680.183	683.827	684.122	682.825				3.644	3.939	2.642

1	2	3	4	5	6	7	8	9	10	11	12
	-					-	-	-			
07		683.534	684.232	684.373	683.751				0.698	0.839	0.217
	08					0.255	0.158	0.584			
08		684.604	684.487	684.531	684.335				-0.117	-0.073	-0.269
	09					0.117	0.073	0.269			
09		684.604	684.604	684.604	684.604				0.00	0.00	0.00
	10					0.051	0.032	0.117			
10		684.604	684.553	684.572	684.365				-0.051	-0.032	-0.117
	11					0.104	0.065	0.239			
11		683.384	684.449	684.507	684.126				1.065	1.123	0.742
	12					0.119	0.074	0.271			
12		680.488	684.330	684.433	683.855				3.842	3.945	3.367
	-					-	-	-			
13		683.537	684.115	684.298	683.360				0.578	0.761	-0.177
	15					0.171	0.107	0.391			
14		682.774	683.944	684.191	682.969				1.17	1.417	0.195
	16					0.007	0.005	0.017			

1	2	3	4	5	6	7	8	9	10	11	12
15		681.555	683.937	684.186	682.952				2.382	2.631	1.397
	17					0.005	0.013	0.012			
16		681.402	683.942	684.193	682.964				2.540	2.797	1.562
	18					0.037	0.020	0.087			
17		678.354	683.979	684.219	683.051				5.625	5.865	4.697
	19					0.151	0.092	0.341			
18		678.354	684.130	684.311	683.392				5.776	5.957	5.038
	-					-	-	-			
19		679.268	683.930	684.184	682.950				4.062	4.916	3.682
	22					0.013	0.010	0.025			
20		677.030	683.943	684.194	682.975				6.913	7.164	5.945
	-					-	-	-			
21		679.268	683.764	684.080	682.560				4.496	4.812	3.292
	25					0.076	0.047	0.171			
22		678.354	683.688	684.033	682.389				5.334	5.679	4.035
	26					0.015	0.009	0.033			

1	2	3	4	5	6	7	8	9	10	11	12
23		676.677	683.673	684.024	682.356				6.996	7.347	5.679
	27					0.003	0.002	0.009			
24		674.390	683.676	684.026	682.364				9.286	9.636	7.974
	28					0.022	0.015	0.051			
25		674.695	683.698	684.041	682.415		0.039		9.003	9.346	7.720
	29					0.060		0.136			
26		675.915	683.758	684.080	682.551				7.843	8.165	6.636

Operational pressure heads:

From Eq. (4.4) the operational pressure heads at feeding point are computed as follows:

Variant f1 = 0.117 + 6.0 = 6.117m

Variant f2 = 0.073 + 6.0 = 6.073m

Variant f3 = 0.269 + 6.00 = 6.269m

The computed operational pressure heads at other nodes are also tabulated in Table 5.19.

Changes in pressure head due to changes in consumption rate ΔH^n_{opr}

Changes in distribution of pressure head at nodes are computed as follows:

$$\Delta H^n_{opr} = \frac{H^n_{opr\ F1} - H^n_{opr\ fi}}{H^n_{opr\ F1}} \quad .. \quad .. \quad (5.3)$$

where $H^n_{opr\ F1}$ = Operational pressure head at node n of variant F1 in problem A

$H^n_{opr\ fi}$ = Operational pressure head at node n of variant fi is problem B

where i = 1, 2, 3.

The computed results are also tabulated in the Table 5.19.

TABLE 5.19: PERCENTAGE OF CHANGES IN OPERATIONAL PRESSURE HEAD AT NODE FOR ALL PIPES VARIANTS IN PROBLEM B OF ADDA F PIPE N.J.T., C.P.K

Node No.	Operational pressure heads H^n_{opr} (m)				Differences in pressure head (m)			Percentage of increase in pressure head		
	q = 0.15	q = 0.13	q = 0.1	q = 0.2	q = 0.13	q = 0.1	q = 0.2			
1	2	3	4	5	6	7	8	9	10	11
01	8.100	8.250	8.420	7.675	0.150	0.320	-0.425	1.9	4.0	-5.2
02	9.217	9.520	9.863	8.366	0.303	0.646	-0.057	3.3	7.0	-9.3
03	10.428	10.732	11.077	9.568	0.304	0.649	-0.860	2.9	6.2	-8.2
04	14.109	14.408	14.746	13.266	0.299	0.637	-0.843	2.1	4.5	-6.0
05	11.492	11.760	12.062	10.738	0.268	0.570	-0.754	2.3	5.0	-6.6
06	9.538	9.761	10.012	8.911	0.223	0.474	-0.627	2.3	5.0	-6.6
07	6.729	6.815	6.912	6.486	0.086	0.183	-0.243	1.3	2.7	-3.6
08	6.000	6.000	6.000	6.000	0.000	0.000	0.000	0.0	0.0	0.0
09	6.157	6.117	6.073	6.269	-0.040	-0.084	0.112	-0.6	-1.4	1.8
10	6.089	6.066	6.041	6.152	-0.023	-0.048	0.063	-0.4	-0.8	1.0

1	2	3	4	5	6	7	8	9	10	11
11	7.169	7.183	7.196	7.011	0.013	0.027	-0.158	0.0	0.0	-2.2
12	9.906	9.959	10.018	9.636	0.053	0.112	-0.270	0.5	1.1	-2.7
13	6.519	6.95	6.834	6.092	0.126	0.265	-0.477	1.9	4.0	-7.3
14	7.104	7.287	7.490	6.464	0.180	0.386	-0.640	2.5	5.4	-9.0
15	8.313	8.499	8.704	7.666	0.186	0.391	-0.647	2.2	4.7	-7.8
16	8.473	8.657	8.870	7.831	0.184	0.397	-0.642	2.2	4.7	-7.6
17	11.570	11.742	11.938	10.966	0.172	0.368	-0.604	1.5	3.2	-5.2
18	11.771	11.893	12.030	11.307	0.122	0.259	-0.464	1.0	2.2	-3.9
19	10.591	10.779	10.989	9.957	0.188	0.398	-0.634	1.8	3.8	-6.0
20	12.846	13.030	13.237	12.218	0.184	0.391	-0.628	1.4	3.0	-4.9
21	10.369	10.613	10.885	9.561	0.244	0.516	-0.808	2.4	5.0	-7.8
22	11.181	11.451	11.752	10.304	0.270	0.571	-0.877	2.4	5.1	-7.8
23	12.838	13.113	13.420	11.948	0.275	0.582	-0.890	2.1	4.5	-6.9
24	15.130	15.403	15.709	14.243	0.273	0.579	-0.887	1.8	3.8	-5.7
25	14.855	15.120	15.419	13.989	0.265	0.564	-0.866	1.8	3.8	-5.8
26	13.715	13.960	14.238	12.905	0.245	0.523	-0.810	3.8	3.8	-5.9

A summary of the relevant changes in operational pressure heads H^n_{opr} with respect to changes in rate of consumption q(gpm) are as follows:

Table 5.20 Maximum changes in operational pressure heads due to changes in consumption rate:

q(gpm) (Form To)	Changes in H^n_{opr} (Varying between)
2.0 - 1.7	-0.6% (node 9) — 3.3% (node 2)
2.0 - 1.3	-1.4% (node 9) — 7.0% (node 2)
2.0 - 2.7	-9.3% (node 2) — 1.8% (node 9)

From the table above, it is observed that the extream changes always accured at both node 2 and 9. Node 9 is tbe feeding point of the network, while node 2 is a service node in the network.

The observed operational pressure heads at feeding point is not a constant for all variants but changes from one to another, to satisfy the minimum pressure head requirements at the nodes in the network.

Increase in consumption rate requires increase of operational pressure head at the feeding point. Decrease of consumption rate allows for decrease in operational pressure head at the feeding point.

On the other hand, a maximum difference of increase in operational pressure head was reflected at the node 2 when consumption rate was reduced. Similarly when the consumption rate was increased, a maximum difference of decrease in operational pressure head was observed at the node No. 2. Node No. 8 was always found as the critical node where the minimum standard pressure head requirements has to be maintained, because of it's highest topo graphical level by nature.

5.7.4.4 Conclusion:

(a) It was possible to locate the critical node on which the maximum differences in operational pressure head was observed.

(b) The maximum changes in operational pressure head due to uniform decrease and increase in consumption rate were achieved are tabulated in Table 5.20.

5.7.5 Solution of problem C

5.7.5.1 Data for hydraulic computation:

Computations were carried out seperately for all the two variants.

Variant F1:

The network diagram of the variant F1 is same as the network diagram of the variant F2.

Variant F2:

The network diagram of the variant f2 is same as the network diagram of the variant F3 of the problem A in residential area C.

5.7.5.2 Results and Analysis:

The error in the operational pressure heads are calculated as follows:

$$\text{Error} = \frac{\left[H^n_{opr}\right]_{F1} - \left[H^n_{opr}\right]_{3i}}{H^n_{opr}{}_{F1}} \quad .. \quad .. \quad (5.4)$$

when $\left[H^n_{opr}\right]_{F1}$ = Operational pressure head at node n in problem A

i = 1 - 2

The computed results are tabulated in Table 5.21.

TABLE 5.21: ERROR IN OPERATIONAL PRESSURE HEAD AT IDENTICAL NODES OF ALL THE VARIANTS IN PROBLEM C OF AREA F

Junction No.			Operation pressure head (m)			Difference in pressure head		Percentage of error	
F1	ℑ1	ℑ2	F1	ℑ1	ℑ2	ℑ1	ℑ2	ℑ1	ℑ2
1	2	3	4	5	6	7	8	9	10
09	09	01	6.157	6.119	6.000	0.038	0.157	0.6	2.5
11	11	02	7.169	7.044	6.850	0.125	-0.319	1.7	4.4
12	12	03	9.906	9.590	9.285	0.316	0.621	3.2	6.3
02	02	04	9.217	9.388	9.641	-0.171	-0.424	-1.9	-4.6
01	01	05	8.100	8.256	8.297	-0.156	-0.197	-1.9	-2.4
07	07	06	6.729	6.816	6.805	-0.087	-0.076	-1.3	-1.1
03	03	07	10.428	10.597	10.843	-0.169	-0.415	-1.6	-4.0
04	04	08	14.109	14.277	14.504	-0.168	-0.395	-1.2	-2.8
05	05	09	11.492	11.557	11.830	-0.165	-0.338	-1.4	-2.9
14	14	10	7.104	6.879	6.438	0.225	0.666	3.2	9.4

1	2	3	4	5	6	7	8	9	10
18	18	11	11.771	11.386	10.380	0.385	1.391	3.3	11.8
22	20	12	11.181	11.100	10.564	0.081	0.617	0.7	5.5
24	22	13	15.130	15.090	14.516	0.040	0.614	0.3	4.1
26	24	14	13.715	13.769	13.116	-0.054	0.599	0.4	4.4

Summary of changes in operational pressure heads with respect to individual scheme are as follows:

Tab. 5.22 Comparison of errors in operational pressure heads for all the two shemes in problem C of area F

Variants	Error in H^n_{opr} (Varying between)
ℑ1	-19% (node 2) - 3.3% (node 18)
ℑ2	-4.4 (node 4) - 11.8 (node 11)

The minimum error occured at node No. 2 of variant ℑ1, which is identical to node 4 of variant ℑ2. Similarly, the maximum error occured at node 18 of variant ℑ1, which is identical to the node 11 of variant ℑ2.

By looking at the percentage of error in operational pressure head at nodes we could conclude that the simplified variant ℑ1 is acceptable but not variant ℑ2.

5.7.5.3 Conclusion:

The conclusion is that some simplification is possible in the pipe network diagram.

5.8 Residential Area G

5.8.1 Formulation of problems:

Here, hydraulic computations were carried out inorder to

(A) estimate the required operational pressure head at feeding point and

(B) to determine the acceptable degree of simplification of network.

5.8.2 Common Data:

Total number of houses = 90

Consumption per house = 0.15 l/s (2.0 gpm)

5.8.3 Solution of problem A

5.8.3.1 Data for hydraulic computation:

Computations were carried out seperately for all three scheme of networks.

Variant G1

The scheme of network with pipe No. junction No. and loop No. are shown in Fig. 5.21.

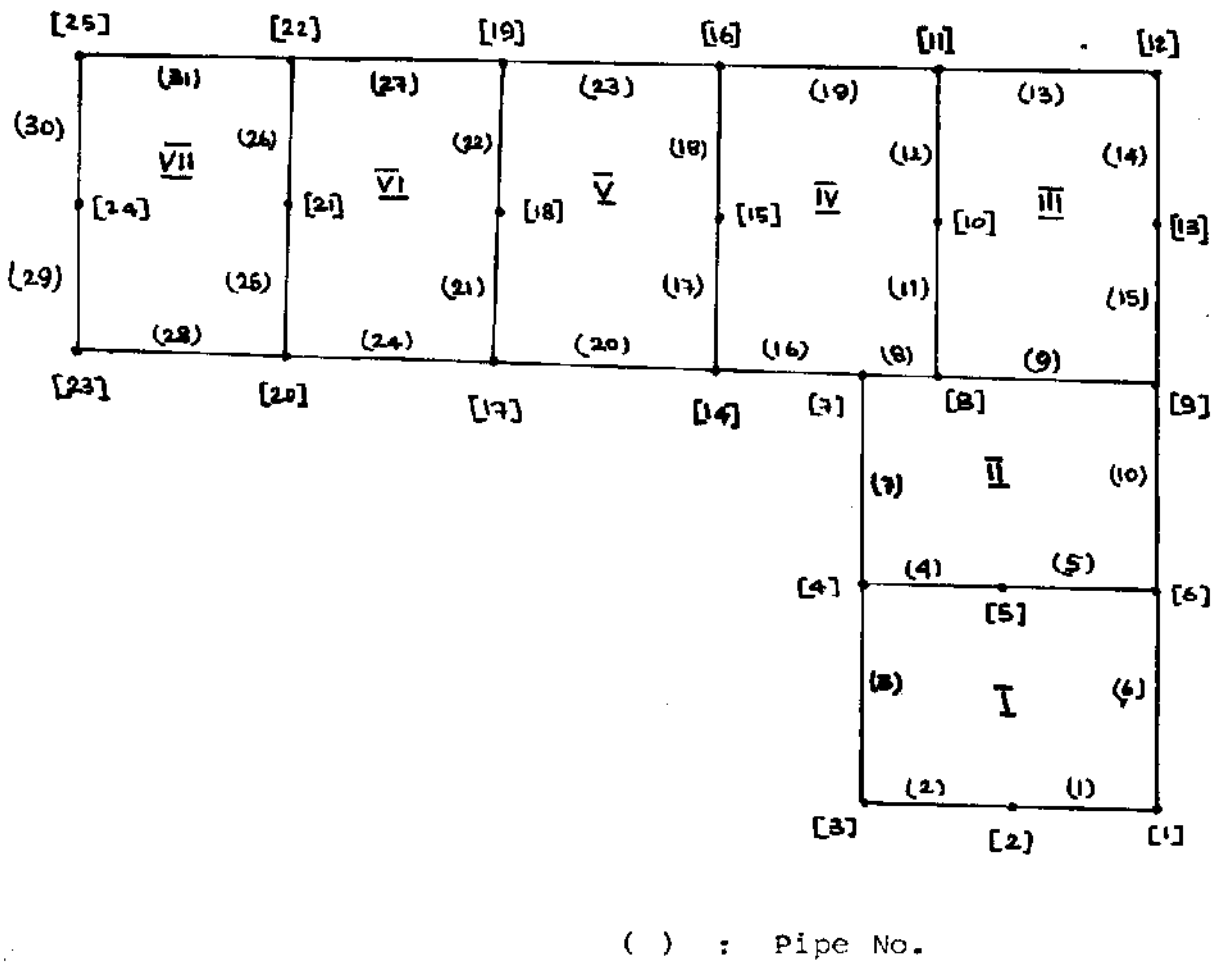
NP = 31 (Number of pipes)

NJ = 25 (Number of junctions)

NL = 07 (Number of loops)

Net external inflow at feeding point equivalent to (90 - 8) houses = 82 houses.

Length and diameter of pipes and flow at nodes are shown in Fig. 5.22

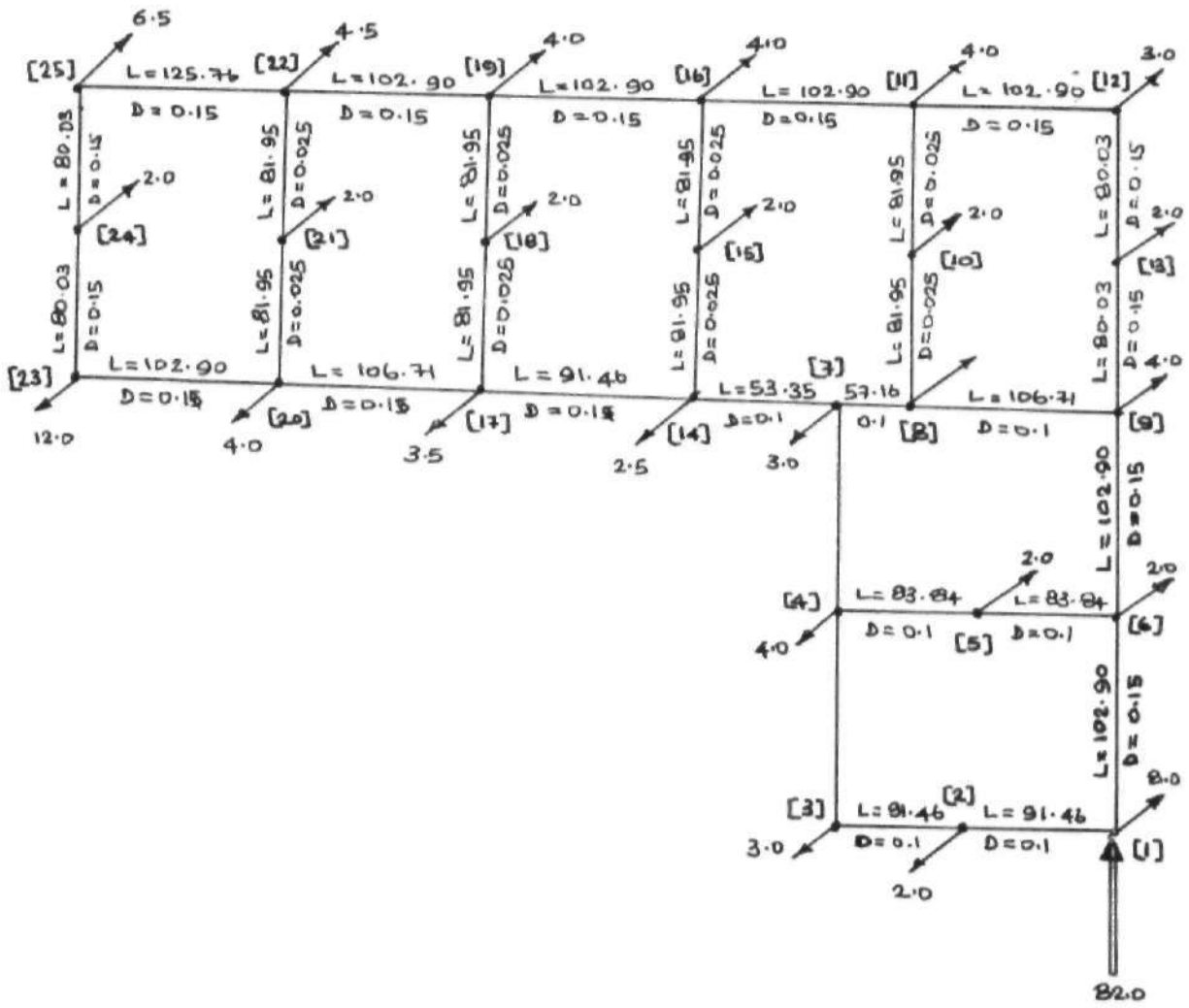


( ) : Pipe No.

[ ] : Junction No.

I,II : Loop No.

FIG. 5.21 PIPE NETWORK DIAGRAM FOR VARIANT G1



L : Pipe Length (m)

D : Pipe Diameter (m)

↗ : No. of Houses at Node

FIG. 5.22 DATA FOR VARIANT G1

Variant G2:

The scheme of the network with pipe No., junction No. and loop No. are shown in the Fig. 5.23.

NP = 23 (No. of pipes)

NJ = 17 (No. of junctions)

NL = 07 (No. of loops)

Net external inflow at feeding point is equivalent to (90 - 9) houses

= 81 houses.

Lengths, diameter of pipes and flowrate at nodes are shown in Fig. 5.24.

Variant G3:

The scheme of the network with pope No., junction No. and loop No. are shown in Fig. 5.25.

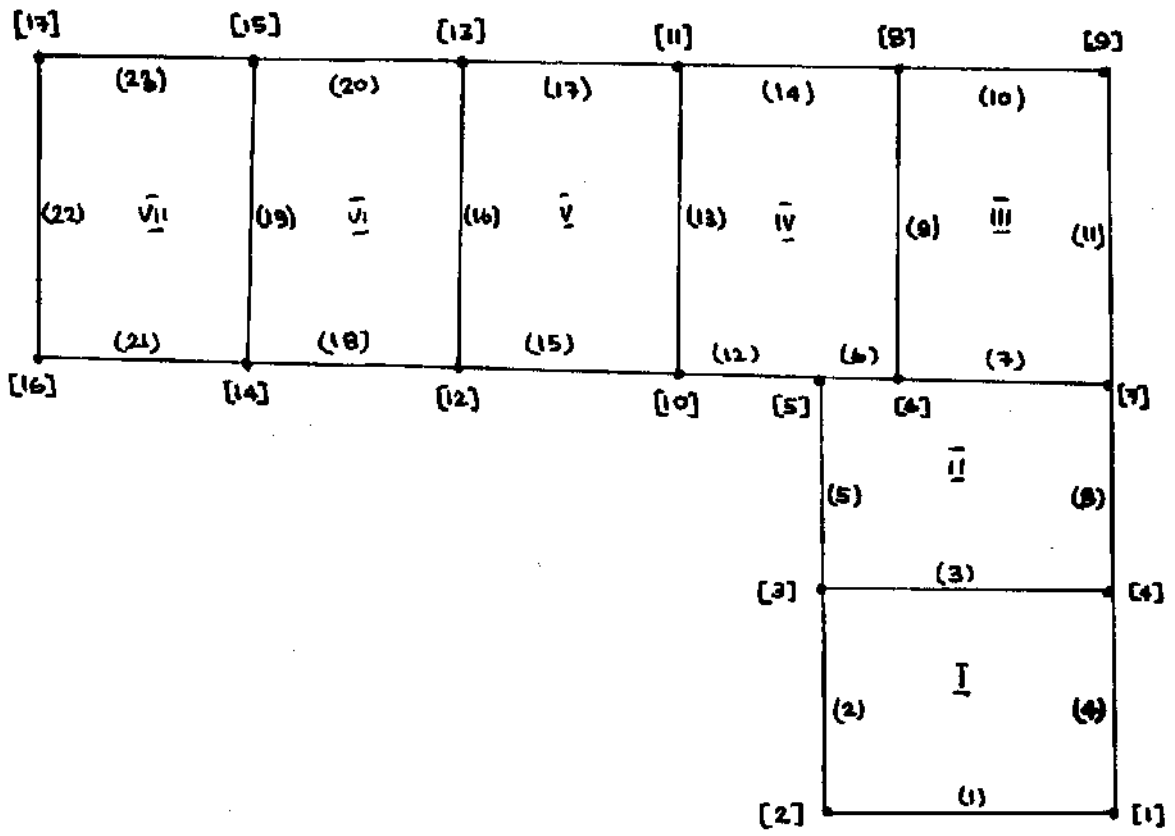
NP = 16 (No. of pipes)

NJ = 15 (No. of junctions)

NL = 02 (No. of loops)

Net external inflow at feeding point is equivalent to (90 - 9) houses = 81 houses.

Lengths, and diameter of pipes and flow at nodes are sown in Fig. 5.26.

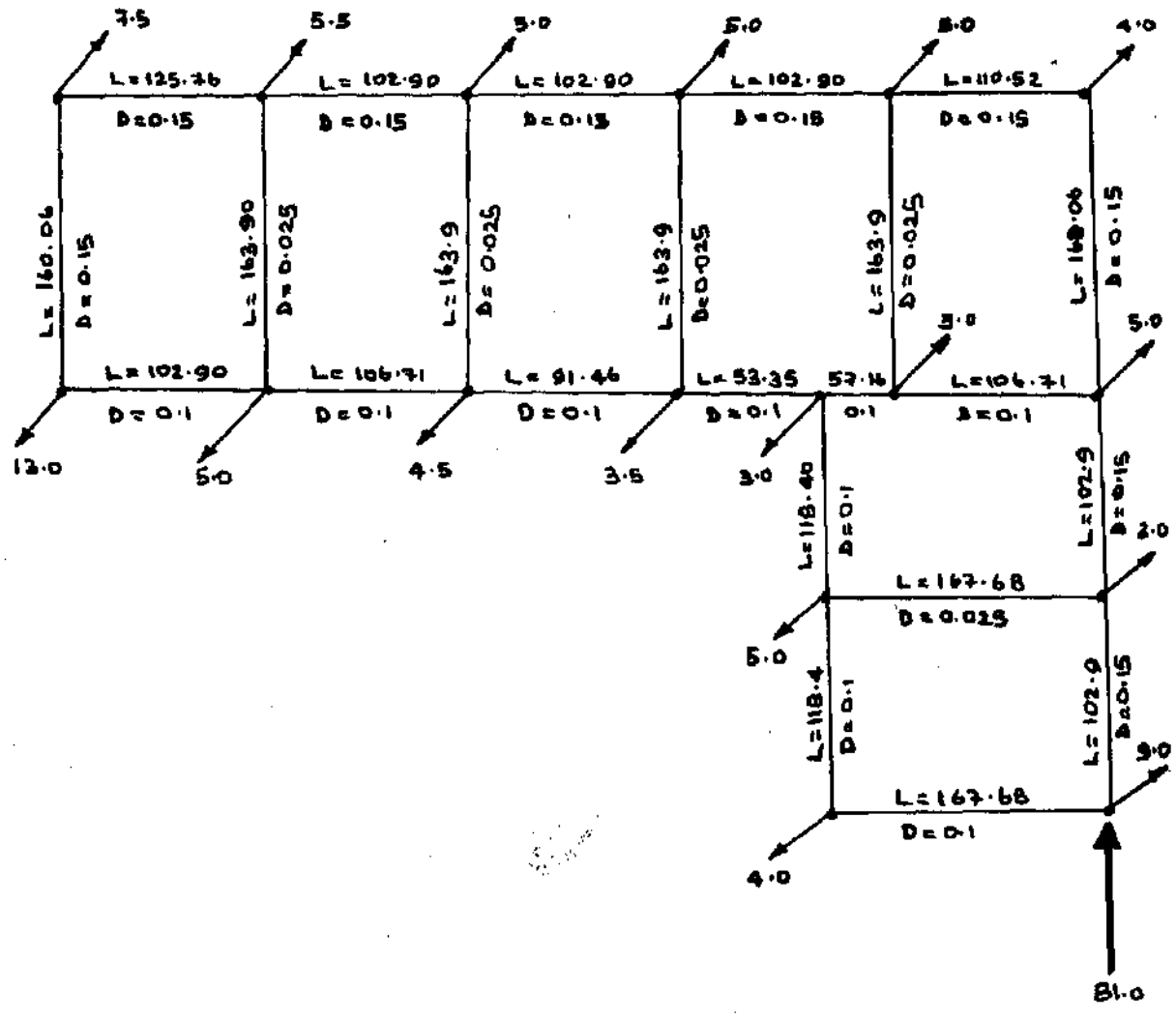


( ) : Pipe No.

[ ] : Junction No.

I,II : Loop No.

FIG. 5.23 PIPE NEWTWORK DIAGRAM FOR VARIANT G2

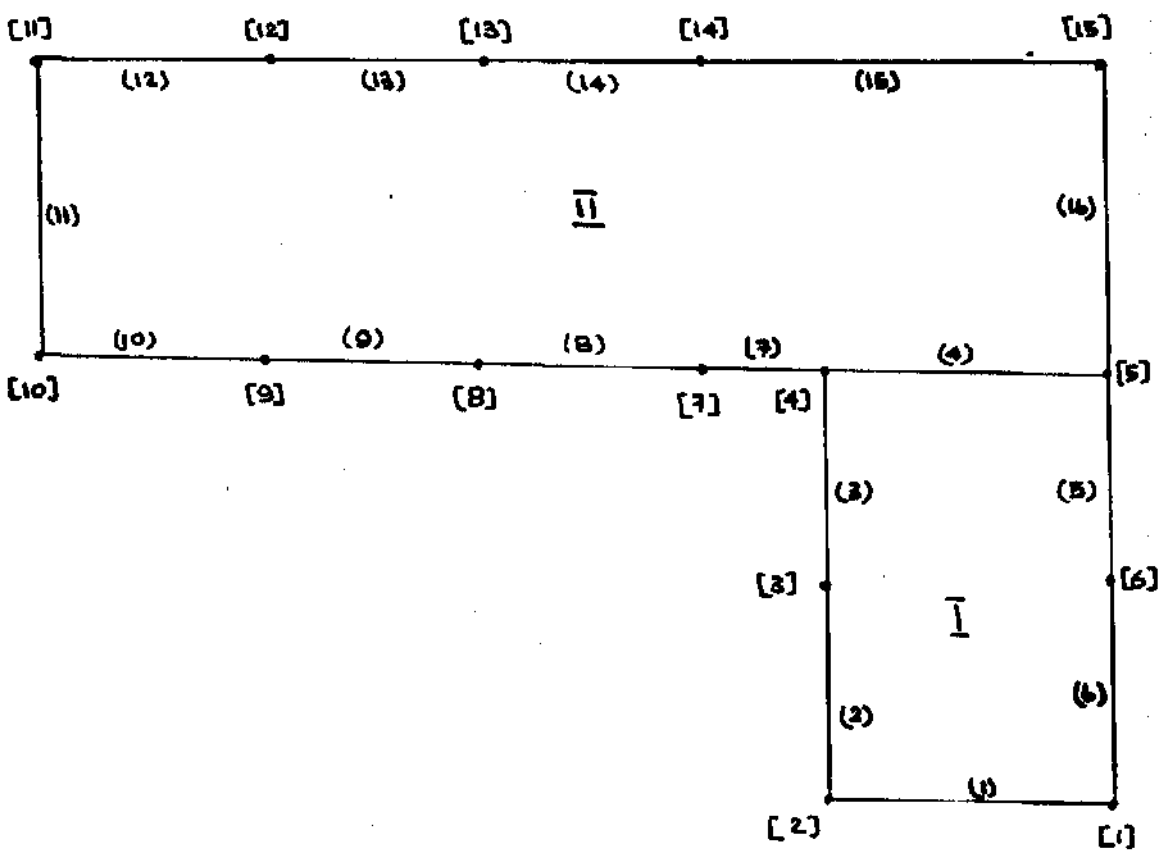


L : Length of the pipe (m)

D : Diameter of the pipe (m)

/ : No. of Houses at node

FIG. 5.24 DATA FOR VARIANT G2

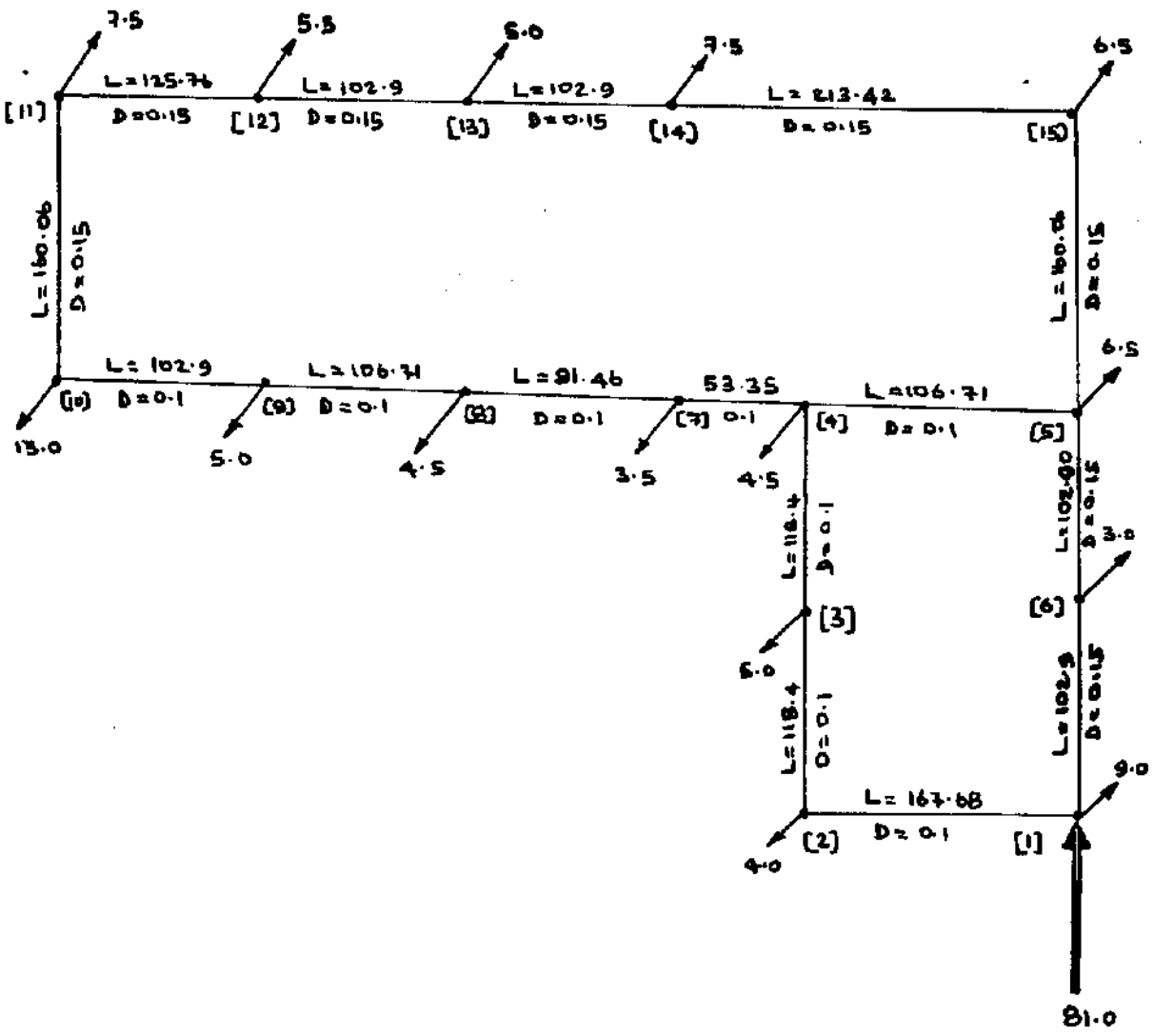


( ) : Pipe No.

[ ] : Junction No.

I,II : Loop No.

FIG. 5.25 PIPE NETWORK DIAGRAM FOR VARIANT G3



L : Pipe Length (m)

D : Pipe Diameter (m)

↗ : No. of Houses at Node

FIG. 5.26 DATA FOR VARIANT G3

5.8.3.2 Results and analysis:

Computations were carried out without any problem for variant G1 and G3. The flowrates are as shown in Fig. 5.27 and 5.28.

For variant G2, the computer reported that "real devided by zero" and the program was terminated without any result, due to zero elements or element encountered as the diagonal elements in the matrix 'a' on the process of solving the linear system of equations for unknown vector 'Q'.

Relative pressure heads:

In the first step, (from Eq. 4.3) the relative pressure heads H_r^n were computed as in Tables 5.23 and 5.24 two variants (G1 and G3). The minimum relative pressure heads were observed at the same node for both variants, (namely, node 23 for variant G1 and node 10 for variant G3), are as follows:

For variant G1 $H_r^{23} = -1.366m$

For variant G3 $H_r^{10} = 1.382m$

5.8.3.2 Results and analysis:

Computations were carried out without any problem for variant G1 and G3. The flowrates are as shown in Fig. 5.27 and 5.28.

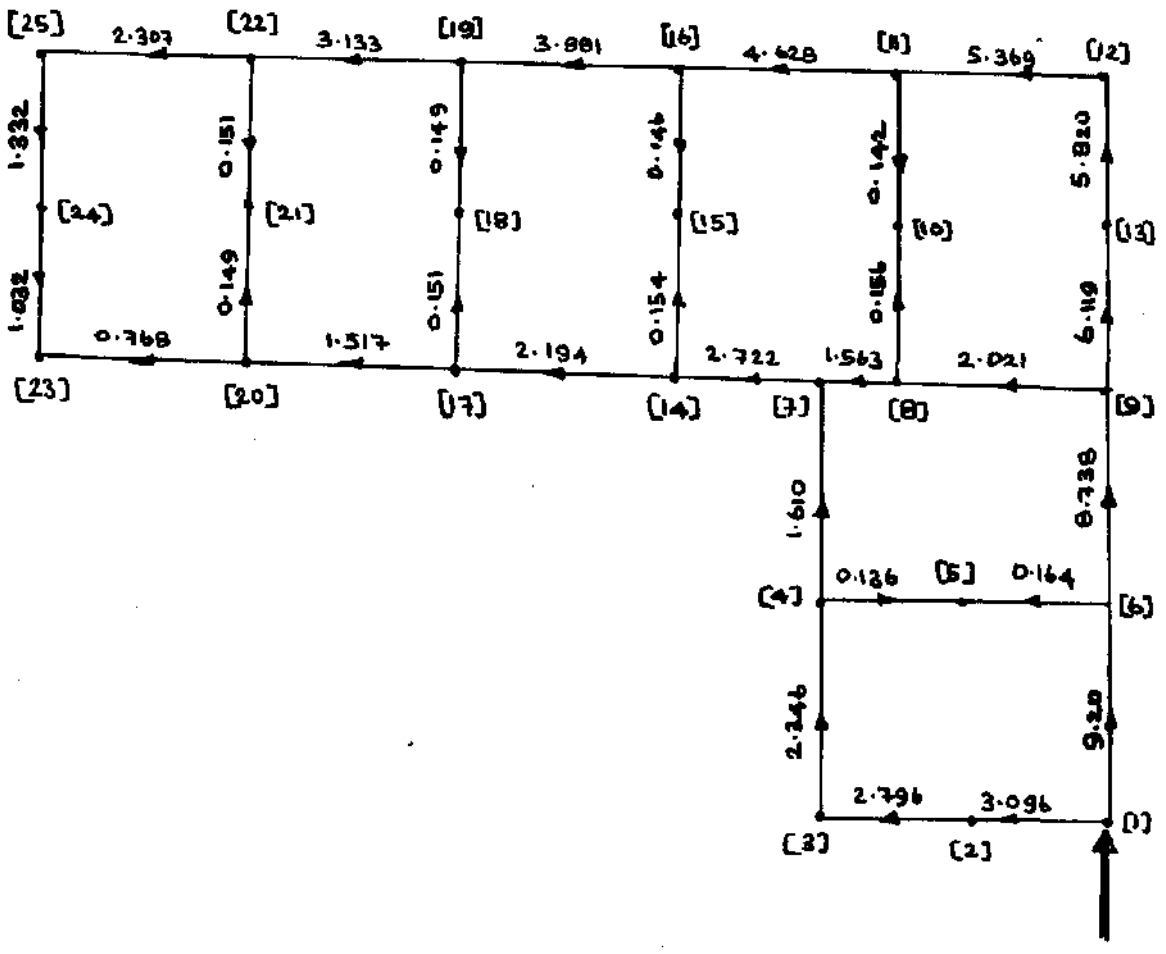
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For variant G1 $H_r^{23} = -1.366m$

For variant G3 $H_r^{10} = 1.382m$


[25]
2·307
[22]
3·133
[19]
3·881
[16]
4·428
[11]
5·369
[12]
1·332
[24]
1·032
0·151
0·149
[21]
0·149
0·151
[18]
0·148
0·154
[15]
0·142
0·156
[10]
5·820
6·119
[13]
0·768
1·517
2·194
2·722
1·563
2·021
[23]
[20]
[17]
[14]
[7]
[8]
[9]
1·610
8·738
0·136
[4]
[5]
0·164
[6]
2·346
9·220
2·796
3·096
[3]
[2]
[1]

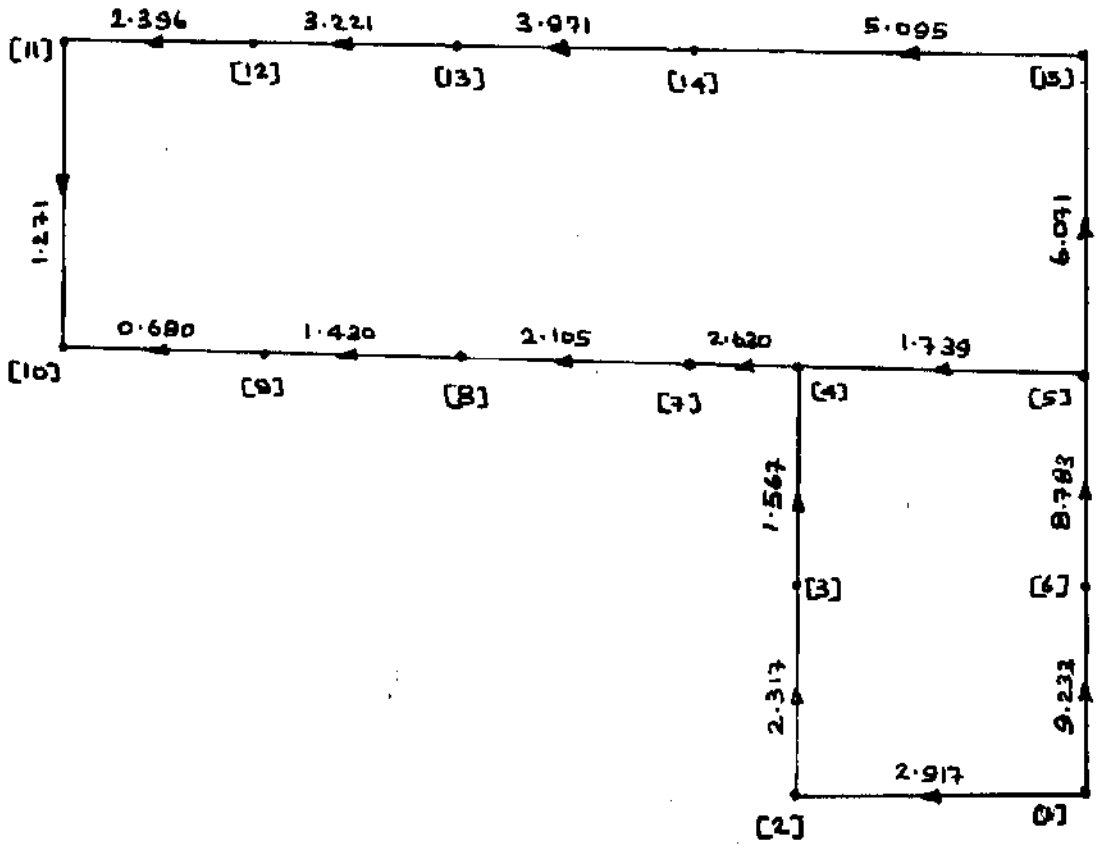


FIG. 5.28 FLOWRATE IN PIPES (1/s) FOR VARIANT G3

TABLE 5.23 THE RELATIVE PRESSURE HEADS AT NODES FOR VARIANT G1

JN No.	Pipe No.	Topo Levels (m)	H.G.L. elevation H_e^n (m)	Fric. head losses in pipes (m)	H_r^n
1	2	3	4	5	6
01		693.902	693.902		0.00
	01			0.109	
02		691.845	693.793		1.948
	02			0.091	
03		689.241	693.701		4.460
	03			0.087	
04		690.244	693.615		3.371
	04			0.363	
05		692.317	693.251		0.934
	05			0.514	
06		693.201	693.765		0.564
	-			-	
07		691.637	693.573		1.941
	08			0.021	
08		692.412	698.594		1.182
	09			0.060	
09		694.250	693.654		-0.596
-	-			-	
10		692.668	693.128		0.460
	12			0.387	
11		692.826	693.515		0.689
	13			0.051	

1	2	3	4	5	6
12		694.838	693.566		-1.272
	14			0.042	
13		694.512	693.608		-0.904
	-			-	
14		690.549	693.522		2.973
	17			0.448	
15		691.509	693.074		1.565
	18			0.404	
16		691.811	693.478		1.667
	-			-	
17		690.470	693.462		2.992
	21			0.431	
18		690.780	693.031		2.251
	22			0.420	
19		691.290	693.451		2.161
	-			-	
20		692.073	693.462		1.389
	25			0.422	
21		692.683	693.040		0.357
	26			0.429	
22		692.415	693.469		1.054
	-			-	
23		694.817	693.451		-1.366
	29			0.002	

1	2	3	4	5	6
24		694.268	693.453		-0.815
	30			0.003	
25		693.659	693.456		-0.203

TABLE 5.24: THE RELATIVE PRESSURE HEADS AT NODES FOR VARIANT G3

JN No.	Pipe No.	Topo Levels (m)	H.G.L. elevation H_e^n	Fric. head losses in pipes (m)	H_r^n (m)
1	2	3	4	5	6
01		693.902	693.902		0.000
	01			0.197	
02		689.241	693.705		4.464
	02			0.085	
03		690.244	693.620		3.376
	03			0.090	
04		691.637	693.580		1.943
	04			0.071	
05		694.250	693.651		-0.599
	05			0.113	
06		693.201	693.764		0.563
	-			-	
07		690.549	693.532		2.983
	08			0.055	
08		690.470	693.477		3.007
	09			0.033	
09		692.073	693.444		1.371
	10			0.009	
10		694.817	693.435		-1.382
	11			0.006	
11		693.659	693.411		-0.218

1	2	3	4	5	6
	12			0.014	
12		699.415	693.455		0.040
	13			0.019	
13		691.290	693.474		2.184
	14			0.028	
14		691.811	693.502		1.691
	15			0.089	
15		694.838	693.591		-1.247

Frictional head losses in supply line to A.E.R.L.C. 8 houses:

The supply line to the above houses were not included in the Figure which shows the pipe network of variant G1. This is because only the closed loops were considered in the linear theory method for analysis of pipe network.

The supply line to these houses is being tapped from node 23 is shown in Fig. 5.29 along with lengths between the nodes, and discharges in the pipes.

The diameter of the supply line is 0.1m

Consumption rate per house q = 0.15 l/s

The Reynold number Re

Eq. (4.8)

$$Re = \frac{4}{\pi D \nu} \cdot Q = 1.27 \times 10^7 \; Q$$

The relative roughness

$$\frac{E}{D} = 0.000025$$

The frictional head losses in the pipe h_f

Eq. (4.10)

$$h_f = 8.279 \times 10^3 \; fLQ^2$$

The computed values of Reynold number, frictional factor, and frictional head losses are tabulated in Table 5.25.

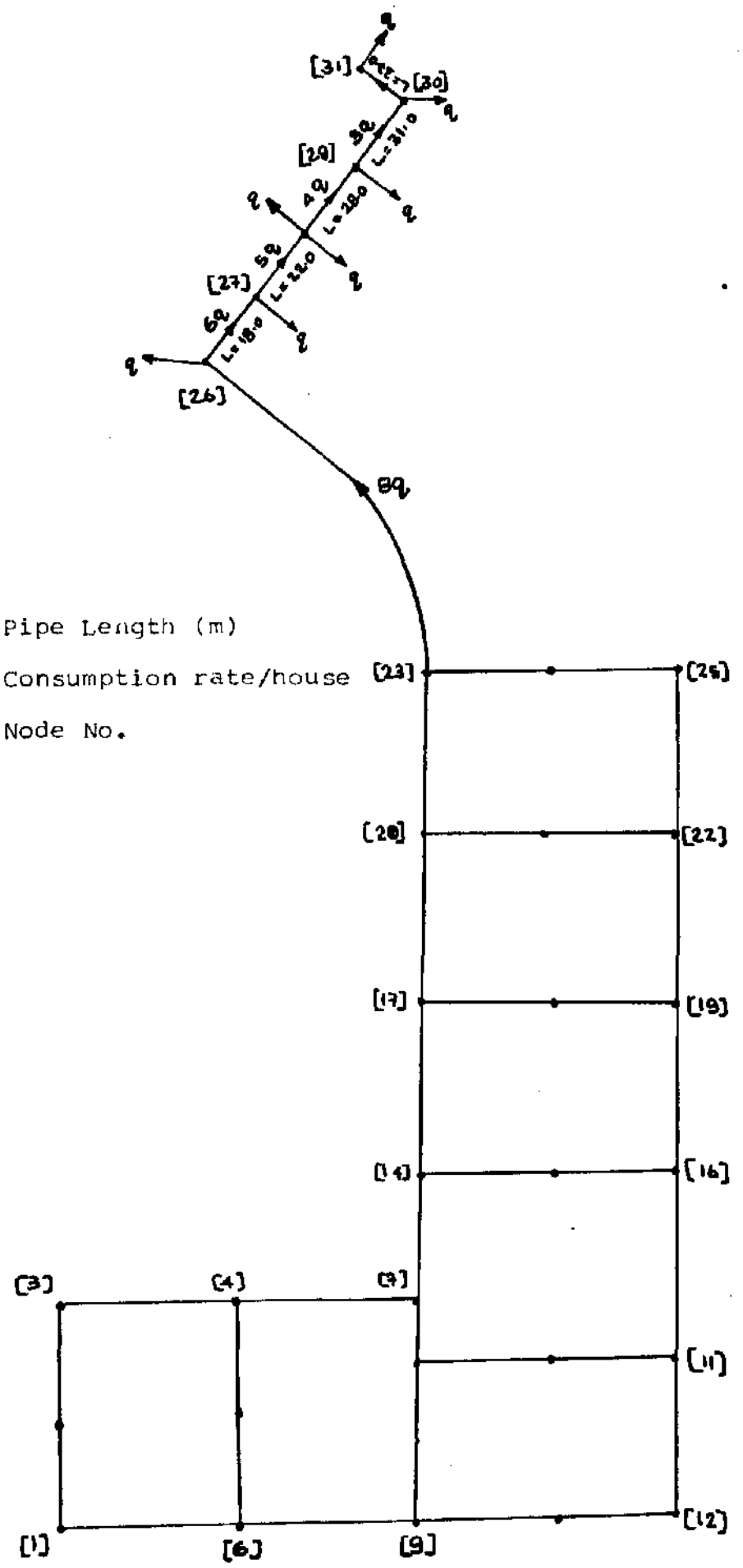


FIG. 5.29 SUPPLY LINE TO AERLS HOUSES FROM NODE NO. 23 OF THE NETWORK DIAGRAM OF VARIANT G1

TABLE 5.25: FRICTIONAL HEAD LOSSES IN SUPPLY LINE WHICH SUPPLIES WATER TO AERLS HOUSES

Node No. (between)	Reynold No. $\times 10^4$	Friction factor f	Fric. head losses h_f (m)
23 - 26	1.524	0.027	0.075
26 - 27	1.143	0.029	0.004
27 - 28	0.953	0.0295	0.003
28 - 29	0.762	0.031	0.003
29 - 30	0.572	0.032	0.002
30 - 31	0.191	0.0305	0.0002
Total Friction head losses between node 23 and 31 =			0.0872m

Operational pressure head at feeding point:

For variant G1

From Table 5.23 the H.G.L. elevation at node 23 when feeding pressure head is zero

= 693.451m

From previous subsection total friction head losses in pipe from node 23 to the last house in AERLS Quarters = 0.087m

Hence, H.G.L. elevation at the last house, when The feeding pressure head is zero = 093.451 - 0.087

= 693.364m

Topo level of the node at the last house

= 700.915m

Hence, Relative pressure head H_r^n at the last house

= 693.364 - 700.915

= -7.551m

minimum standard pressure head for single storey building = 6m

Hence, the required operational pressure head at feeding point = 6.0 + 7.551m

= 13.551m

For variant G3

From Table 5.24 the H.G.L. elevation at node 10 when the feeding pressure head is zero

= 693.435

Friction head losses bet. the node 10 and last house = 0.087m

Hence, H.G.L. elevation at the last house when the feeding pressure head is zero = 693.435 - 0.087

= 693.348m

The topo level of the node at the last house

= 700.915m

Hence, relative pressure head H_r^n = 693.384.- 700.915

= -7.567m

The minimum standard pressure head to be maintained at single storey house. = 6.0m

Hence, the operational pressure head required at the feeding point = 6.0 + 7.567

= 13.567m

Check on operational pressure head at nodes feeding two storey buildings:

From the previous result, The operational pressure head at the feeding point for the variant G1 is 13.551m

The operational pressure head at the nodes which supply water to two storey residential houses along with the relative pressure headsare tabulated in Table 5.26.

Table 5.26 Operational pressure head at nodes feeding two storey houses in area G

Node No.	H_r^n (m)	H_{opr}^n (m)
09	-0.596	12.955
12	-1.272	12.279
13	-0.904	12.647

5.8.3.3 Conclusions:

The required pressure head at the feeding point to maintain the minimum required pressure heads in the network is = 13.551m.

5.8.4 Solution of problem B

Here, the operational pressure heads at similar nodes in previous subsection 5.8.3 were compared. The operational pressure heads are tabulated in Table 5.27.

5.4.4.1 Changes in operational pressure heads H_{opr}

$$H_{opr} = \frac{\left[H^n_{opr}\right]_A - \left[H^n_{opr}\right]_B}{\left[H^n_{opr}\right]_A} \quad \ldots \quad (5.5)$$

where, A denotes variant G1

B denotes variant G3

using the above Eq. 5.5 changes in operational pressure heads were computed. The results are tabulated as in Table 5.27.

5.8.4.2 Conclusions:

The changes in distribution of operational pressure heads at nodes due to simplification of pipe network is so small as 0.33%, that could be neglected in our problem.

TABLE 5.27: ERROR IN OPERATIONAL PRESSURE HEADS DUE TO SIMPLIFICATION OF PIPE NETWORK OF AREA G

JN		Operational pressure heads (m)		Difference in operational pressure head	percentage change
G1	G3	G1	G3		
01	01	13.551	13.567	0.016	0.12
03	02	18.011	18.031	0.020	0.11
04	03	16.922	16.943	0.021	0.12
07	04	15.492	15.510	0.018	0.12
09	05	12.955	12.968	0.013	0.10
06	06	14.115	14.130	0.015	0.11
14	07	16.524	16.550	0.026	0.16
17	08	16.543	16.574	0.031	0.19
20	09	14.940	14.938	0.002	0.01
23	10	12.185	12.185	0.00	0.00
25	11	13.348	13.349	0.001	0.01
22	12	14.605	14.607	0.002	0.01
19	13	15.712	15.751	0.039	0.25
16	14	15.218	15.258	0.040	0.26
12	15	12.279	12.320	0.041	0.33
			The maximum error in operational pressure head =		0.33%

CHAPTER 6

DETERMINATION OF THE HEIGHT OF THE CENTRAL OVER HEAD TANK AND CORRECTION TO THE SUPPLY LINES

6.1 Height of central over head tank:

The results obtained in the previous section as the results of satisfying minimum pressure head conditions at nodes in different residential areas are summarised in Table 6.1.

Table 6.1. Required height of water surface and its supply lines:

Areas	C	E	F	G
Required height of water surface above the bottom of the tank (m)	13.073	2.869	-5.980	11.523
Diameter of supply line (m)	0.2	0.2	0.2	0.2
Length of supply line (m)	110.0	195.0	580.0	1005.0
Discharge (l/s)	19.35	12.3	24.0	13.5

From the table above, the critical area which determines the height of the central over head tank is area C.

The required minimum height of the water surface in the tank above the bottom ground level = 13.073m.

Hence, Let selected minimum height of water surface (height of the tower) = 13.5m

Hence, This gives H.S.L. elevation at the bottom of the tank = 698.171 + 13.5 = 711.671m.

6.2 Corrected diameter of supply lines from the central over head tank:

The selection of the size of supply lines from central over head tank were carried out to satisfy the following conditions:

(a) To satisfy the required operational pressure head at feeding points.

(b) To supply water smulteneously to all residential areas by gravity without any further control on the supply line.

(c) To utilise the existing over head tanks.

Computations were carried out to design the supply lines for the selected water surface height of 13.5m above the ground. The results are as tabulated in Table 6.2.

It was possible to select a single diameter supply line for each area except for area F, to which two different sizes of pipes were chosen as the supply main.

6.3 Suitability and recommended use of existing over head tanks:

The purpose is to investigate whether it could utilise the existing over head tanks to the respective areas.

TABLE 6.2: REQUIRED DIAMETER AND LENGTH OF SUPPLY LINE AND OPERATIONAL PRESSURE HEAD AT FEEDING POINT WHEN SUPPLIED FROM CENTRAL OVER HEAD T A N K

	Residential Areas	C	E	F		G
1	H.G.L. elevation at bottom of the central over head tank (m)	711.671	711.671	711.671		711.671
2	Rate of supply (l/s)	19.35	12.3	24.3		13.5
3	Length of supply line L (m)	110.0	195.0	120	460	1005.0
4	Diameter of supply line D (m)	0.2	0.1	0.1	0.15	0.2
5	Reynold No. Re Eq.(4.8)	1.22×10^5	1.56×10^5	3.045×10^5	2.03×10^5	8.5×10^4
6	Relative roughness $(\frac{E}{D})$	0.0000125	0.000025	0.000025	0.0000167	0.0000125
7	Friction factor f	0.017	0.0162	0.014	0.0153	0.018
8	Frictional head losses in supply line h_f Eq. (9.10)	0.176	3.96	12.321		0.85m
9	H.G.L. elevation at feeding point (m)	711.495	707.711	699.35		710.821
10	Operational pressure head at feeding point H^n_{opr} (m)	17.593	14.217	14.746		16.919

Fig. 6.1 shows the location of the feeding point of individual areas with the location of the tapping point which supplies water to the existing over head tank.

6.3.1 Check on pressure head requirements at the tapping points to feed the existing over tank:

For area F tank:

Tapping point and the feeding point in area F is same namely the node No. 9 in Fig.6.1A

For common tank at areas E and G:

The feeding point is node No. 15. The tapping point is node No. 8 in Fig. 6.1A

The computed results are tabulated as shown in Table 6.3. A single size of supply main is used for area F tank while two sizes for the other tank.

Conclusion:

From Table 6.3, since the operational pressure head at the tapping points are higher than the required minimum pressure head, we could conclude that the existing tanks could be fed from the tapping point of the respective networks.

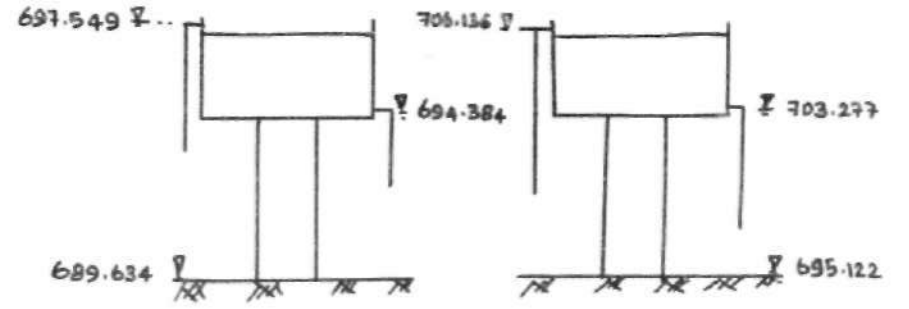


FIG. 6.1B LEVEL OF EXISTING TANKS

FIG. 6. 1A LOCATIONS OF FEEDING POINTS AND TAPPING POINTS

TABLE 6.3: DERIVATION OF OPERATIONAL PRESSURE HEAD AT TAPPING POINTS OF EXISTING OVER HEAD TANK, WHEN THE NETWORK FED FROM CENTRAL OVER HEAD TANK

	Descriptions	For Area F tank	For the tank common to area E & G	
1.	Length of supply line from tapping point to the inlet of the tank (m)	76.89	7.93	150.0
2.	Diameter of supply lines	0.15	0.1	0.15
3.	Rate of flow in supply line (l/s)	2.4	12.3	
4.	Reynold No. Re	2.03×10^5	1.56×10^5	1.04×10^5
5.	Relative roughness $\frac{E}{D}$	0.0000167	0.000025	0.0000167
6.	Friction factor f	0.0153	0.0162	0.0172
7.	Frictional losses in supply line (m)	0.735	0.161	0.472
8.	Elevation of the inlet of the tank (m) see Fig. 6.1 3	697.549	706.136	
9.	Minimum required H.G.L elevation at tapping point to supply water to the tank (raw 7 and raw 8)	698.284	706.769	
10.	Topo level at tapping point, Tables 5.7 and 5.14	684.604	692.799	
11.	Minimum required pressure head at tapping point (raw 9 - 10) (m)	13.680	13.970	
12.	H.G.L. elevation at tapping point when supply from central over head tank (Tab.6.2) & 5.7)	699.35	707.085	
13.	Operational pressure head at tapping point when supplied from central over head tank	14.746	14.286	

6.3.2 Check on pressure head requirement at feeding points when supplied from individual tanks:

(a) For area F tank:

Minimum water surface elevation (Fig. 6.1B)

= 694.384m

Frictional head losses in supply line to feeding point from Table 6.3 = $\frac{73.725}{76.89}$ x 0.7.35 = 0.705m

Hence, minimum H.G.L. elevation at feeding point

= 694.384 - 0.705 = 693.679m

Hence, minimum operational pressure head at feeding point = 693.679 - 684.604 = 9.075m

From subsection 5.7.3.2, the required operational pressure head at feeding point = 6.157m

Hence, we could conclude that the tank height is more than sufficient to satisfy the minumum pressure head requirement at all nodes in area F.

(b) Common tank at areas E and G

Minimum water surface elevation in the tank (Fig. 6.1B) = 703.277m

frictional head loss in supply line from tank to the feeding point (Table 6.3),

to Area G = $\frac{5.071}{7.930}$ x 0.161 + $\frac{28.9}{150}$ x 0.472

= 0.194m

to Area E = $\frac{5.071}{7.930}$ x 0.161 + 0.472 = 0.575m

Hence, minimum H.G.L. elevation at feeding point

to area G = 703.277 - 0.194 = 703.083m

to area E = 703.277 - 0.575 = 702.702m

Hence, operational pressure head at feeding point

to area G = 703.083 - 693.902 = 9.181m

to area E = 702.702 - 692.799 = 9.903m

From subsection 5.8.3.2 and 5.6.3.2, the operational pressure head required to maintain the standard minimum pressure head at nodes in the network are as follows:

to area G = 13.551m

to area E = 7.402m

Hence, we could conclude that the common tank at E and G could fully satisfy the minimum standard pressure head requirements at all nodes in area E. But in the case of area G (Reference to Table 5.23) it can satisfy the requirements in all nodes except the nodes feeding the two storey buildings on the other hand it is impossible to meet the requirements in AERLS houses.

6.4 Conclusions:

- Design height of the tower for central over head tank is 13.5m above the ground level
- Design of supply mains from central over head tank is derived as follows:

Table 6.4 Diameter and length of supply mains from central over head tank:

Areas	C	E	F		G
Diameter D (m)	0.2	0.1	0.1	0.15	0.2
Lengths L (m)	110.0	195.0	120.0	460.0	1005.0

- The existing tank at area F and the common tank at E and G could be utilised for area F and E respectively to fully satisfy the minimum pressure head requirement at all nodes.

CHAPTER 7

DETERMINATION OF THE CAPACITY OF THE CENTRAL OVER HEAD TANK

7.1 Basic requirements:

The reservoir capacity should be sufficient enough to equalise the rate of flow in and out of the tank in peak period of consumption and also for any emergencies such as Fire fightings and unexpected interuptions of supply in pumping mains and treatment plant.

Hence, the storage capacity of any service reservoir should be determined based on the results obtained from a Mass diagram of water consumption or from a plot of hourly water consumption rate of the area under consideration along with estimated known provisional capacity for fire flow requirements and the period of (constant rate) pumping. Usually, the capacity required for equalising daily flow requirement varies between $\frac{1}{6}$th to $\frac{1}{3}$rd of the daily demand [Layla et al (1978)] but an additional quantity has to be added in computing the capacity of the service storage tank to meet the demand of any emergencies.

According to B.S.S. (British standard of specification) for municipal services, requires minimum of 40% of daily demand as the recommended capacity of any service reservoir.

7.2 Estimation of daily demand:

Total No. of houses in high pressure zone (E,F,G,C) = 82 + 160 + 90 + 129

= 461 houses

Hence, estimated daily demand = 0.15 l/s x 461

= 69.15 l/s

Hence, estimated daily demand/day = 5,974,560 l

Hence, mininum required capacity of the central over head tank = 0.4 x 5,974,560 l

= 2,389,824 l

= $2,290m^3$

7.3 Conclurion:

The selected capacity of the central over head tank = $2300m^3$.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

The followings are the conclusions and recommendations from the analysis of this work.

1. The existing distribution networks of individual residential areas are adequate. Therefore, modifications are not required in all areas under consideration. (section 5).
2. The existing over head tank at area F could serve and maintain the minimum pressure head requirements at all nodes in area F (sub.sec. 6.2.2 a).
3. The existing over head tank common to areas E, G and AERLS houses proved to be incapable of feeding AERLS houses and the entire area G, but it could serve and maintain the minimum pressure head requirements satisfactorily at all nodes in area E. (sub. section 6.2.2 b)
4. A method of chosing the location of best feeding point was developed, considering required height of the tank and length of supply line. The best location of feeding points were found for area C and E. (sub-sections 5.1.3.2.6 and 5.2.3.2.6)
5. It was possible to derive the final diameter of individual supply lines for each areas to supply water by gravity from central over head

tank without any further control in supply line by adjustment of diameter of supply line to height of the central over head tank and the pressure head in the individual residential area. (sub-section 6.2)

6. The selected location of the central over head tank could be concluded as very satisfactory because of the following reasons:

   (a) It requires very reasonably acceptable height of the tank probably due to its highest location within the area of consideration. (sub-section 6.1)

   (b) The location is closest to the critical area C which requires the highest operational pressure head at feeding point. Therefore, the frictional head losses in supply line is very minimum, requires minimum height of the tank. (Fig. 6.1A)

   (c) Becuase the selected location is within the area under consideration, it requires lesser total length of supply lines compared to any other location outside the area.

7. Considering the required large volume of the tank (2300m^3) and relatively high topo level different between the pump house and the location of the central over head tank (45.8m) it is recommended to investigate inorder to cut down the expected cost of the project by shairing the

required volume of the tank in two different tanks: under ground and over head tanks. Water from the treatment plant would be supplied to the ground tank and it will be lifted to the over head tank by additional booster pumps at the bottom of the tank.

8. The maximum error in distribution of pressure head due to minor losses was estimated as 1.91% which could be neglected in the analysis. (sub-section 5.1.4)

9. The maximum error in distribution of pressure head due to increase in roughness of pipe by 500% was estimated as 0.1%. Therefore, any small error in estimated value of roughness of pipe used will not have much effects on distribution of pressure head. (sub-section 5.2.4)

10. As the results of the analysis of changes in consumption rate in area F it was possible to locate the critical node on which the maximum differences in operational pressure head was observed.

11. Uniform decreases of 15% and 35% in consumption rate for area F produces maximum increases of 3.3% and 7.7% respectively in operational pressure heads at the critical node. On the other hand, a uniform increase of 35% in consumption rate produces a maximum decrease of 9.3% in operational pressure head at the critical node. (sub-section 5.3.4)

12. It is worth to advise the authority of the University to investigate the possibility of reducing the consumption rate by introducing water meters and relevant water rate charges, planning for well design Compacted Multi-storied staff residential houses instead of individual and far-apart compounds and large scale gardening within the residential areas. (sub-section 4.2.1)

13. Determination of the influences of simplification by removing some connections and or Redistributing the external out flow at nodes due to house demands on distribution of pressure head is always possible, but to define a criteria for the method of simplification that could be acceptable for the purpose of the analysis was not able to achieved. Therefore, the conclusion is that some simplification is possible. (sub-section 5.4.4,and 5.3.5.

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