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Computer Aided Design of Water Distribution System and Sewer System for Vijayanagara Layout of Mysore City

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Abstract:

The main objective of the study is to design and estimate water distribution network and underground drainage system for a residential layout of Vijayanagara 2nd stage of Mysore city using LOOP and SEWER software. The project area is 7.9 hectares consists of 353 sites. The water supply system is designed for this layout. The reliable source of water supply for this layout is mainly from Vanivilas water supply board. The present study provides estimation of overall cost of the project. The major cost components like pipe cost, manholes, laying of sewer pipe lines, are included in the analysis. Total cost of UGD for the layout is estimated to be (in 1000) ₹ 8817.56.

Keywords:

Distribution System Design, Estimation of Cost, LOOP and SEWER Software, Manhole, Total Cost, Underground Drainage System

1. Introduction:

India is second most populous country in the world after China. 17% of the world population is constituted by India (Chakrabarti and Sarkar, 2011). The change in lifestyle, standard of living of the people demands for excessive land area for residential accommodations and hence increases the usage water. India receives about 4000km^3 of water per year through precipitation (Kumar et al. 2005). Hence it contributes for the water supply to layouts and areas outside cities by the primarily fulfillment of water availability by ground water. The components of water distribution system that are used to transport water from source to the supply end (De Corte and Sörensen 2013). In the last decade numerous research has been carried out in the water distribution system design (Samani and Mottaghi 2013). Increase in population, urbanization and industrialization resulted in discriminate use and pollution of water in the locally available sources, which demanded and led to modern water supply facilities. It required conveyance of water, treatment system to bring available water within the permissible limits of constituents, storage facilities, and distribution systems. These water supply systems are spread over a large area and exposed to polluting surroundings composed of ground water containing excessive quantities of chemicals, discharges from sewers and leachates from waste deposits in the ground. The distribution network requires in-depth analysis for reliability with respect to quality. Extensive research has been carried out to optimize pipeline design assuming a predetermined geographical layout of the distribution system (Lejano 2006). The capacity of network is equally important as quality to deliver required rate of water with sufficient hydraulic head to cover the entire area. Abnormal increase in demand when water supply is limited for intermittent supply periods and the corresponding effects on network requires analysis.

Due to the complexity of the problem, most of the existing algorithms are therefore restricted to considering the hydraulic and availability requirements, leading to the so-called optimal component design of the pipe networks (Afshar

2005). There are integrated approaches has been developed for water distribution system design (Goulter and Morgan 2011). Present situation there are many methods to detect outliers in network flow measurements that may be due to pipe bursts or unusual consumptions are fundamental to improve water distribution system(Loureiro et al. 2015). To avoid these pressure bursts y introducing flow control vales at various strategic locations within the system (Soltanjalili et al. 2010; Ali, 2015). Due to inherent variability in instantaneous water consumption levels, values of demands at nodes in a water distribution system remain one of the major sources of uncertainty in the network design process (Babayan et al. 2014). For distribution network optimization problems, many methods have been developed to solve the problems such as Shuffle frog leaping algorithms etc.,(Eusuff et al. 2006). A successful implementation and operation of a water supply scheme highly depends on having an efficient treatment unit followed by a perfectly designed water distribution system network. However, relentless developments in urban and semi urban areas have made the technical and managerial levels of water distribution and wastewater collection system complex. The similar progress in the software's has made computer driven methods to cope with them imperatively.

In this paper the design of water distribution network has been done using LOOP V 3.0 software and design of underground drainage system design is done using SEWER software. With all demand nodes, resource nodes, adjacency matrix and pipe lengths assumed known; the aim of this phase is to select both the diameter and the material of each pipe in such a way that the total cost of the network is minimized. Many factors are involved in tackling the problem of providing protected water to the community(Threats 2009). The factors are added with the need for cost wise economy which requires a comprehensive analysis of the design parameters. Optimal use of pipe size and material in supply, with adequate pressure and quality necessities are the primary concern of the designer(Chambers et al. 2004). The per capita cost involved in providing a water supply system is important in view of the financial resource constraints.

One of the major problems is the interaction between wastewater and water supply pipelines, which is to be avoided at all costs. Previously the designing of Water Distribution Network and Underground Drainage system were done manually which involved major expenditure for maintenance and operation besides cost of construction. Now-a-days computer models have been developed like LOOP 3.0 version and Water GEMS for Water Distribution Network and in-house excel program sheet for Under Ground Drainage System. With the help of these programs and system analysis the best alternative design with least cost can be arrived.

2. Materials and Methodology:

The main objective of this study is to design and estimate the Water Distribution Network and Under Ground Drainage System for a private residential layout of Vijayanagara area. Specific objectives include sampling and analysis of the groundwater collected at a private residential layout in Vijayanagar, Mysore, collection of field data related to the selected layout, designing a water distribution network for the selected layout using LOOP 3.0, designing an underground drainage system using K.U.W.S. and D.B in-house excel spreadsheet and preparation of cost estimates based on the design data.

2.1 Study Area:

Vijayanagara layout is in Mysore District of Karnataka State, India. It is located 1 KM towards South from District headquarters Mysore. 148 KM from State capital Bangalore. Vijayanagara is surrounded by Shrirangapattana Taluk towards North, Nanjangud Taluk towards South, Pandavapura Taluk towards North, and Tirumakudal-Narsipur Taluk towards East. The Vijayanagara area is in the outskirts of Mysore, and hence is devoid of surface reservoirs. To overcome this limitation, based on the geological survey, availability of ground water is proven to be the best alternative source of water as a result of which, bore-wells are required to be sited.The proposed residential layout is an isolated land located in an extent of 7.9 hectare at sy no. 216(p) 217, Vijayanagara village, Jayapura hobli, Mysore district. It consists of 353 plots. The estimated number of bore-wells is 4. The total water demand is 3.318 LPS.

2.2 Loop Software:

LOOP does simulation of specified water distribution network which is characterized by pipes and nodes. It requires data such as pipe lengths, diameters, friction coefficients, nodal demands, ground elevation and data describing the geometry of the network. With the input data, the program computes flows in pipes, pressures at nodes and the pipe costs. It considers only fixed head reservoirs and not any variable head reservoirs such as pumps.

LOOP 3.0 can handle up to 500 pipes and can simulate up to 400 nodes. It accepts any looped, partially looped / branched or completely branched network and in general it simulates the hydraulic response of a network to a single or multiple input with at least one known hydraulic gradient line elevation. However it does not have any provision to allow display of hydraulic grade lines on screen (Quindry et al., 1981).

2.2.1 Salient Features of the Loop Version 3.0 Include

1. Simulation of a given network under steady state conditions.
2. Design of a new network by selecting suitable pipe sizes from the available set of pipes.
3. Use of Hazen-Williams equation for estimation of hydraulic gradient.
4. Uses the Hardy-Cross method of analysis to balance the flow and pressures in distribution system.
5. Design offers a starting solution only to be improved by the design to make it more cost-effective. The pipe size selection algorithm is based on hydraulic gradient line to ensure minimum terminal head at nodes.

$$H_s - h_L = h_{\min} \qquad (1)$$

H_s= Source head, h_L= head loss obtained by equations, $h_{\min}$= terminal head loss.

2.3 Development of Self-Cleansing Velocity Parameters:

It has described the procedure for determination of self-cleansing condition in partially full sewers. The major portion of the procedure described is that the coverage flow velocity should be sufficient to provide for the scour of deposits under minimum flow conditions. It has been suggested that 0.6m/s is sufficient velocity under half and full flows, and velocities of 0.3 to 0.4 m/s are suggested for less than half full conditions. A formula has been developed for the computation of appropriate sewer slopes to provide the suggested self-cleansing velocities (Bong 2014)(Pei-Te Chiueh 1999). For convenience, the minimum velocities for various ranges of pipe diameters are presented in Table 1(Khudenko n.d.) Equation for Self Cleansing velocity mentioned by,

$$V_s = \sqrt{\frac{8K}{f'}(S_s - 1)gd'} \qquad (2)$$

K = constant, for clean inorganic solids = 0.04 and for organic solids = 0.06, f' = Darcy Weisbach friction factor (for sewers = 0.03), S_s = Specific gravity of sediments, g = gravity acceleration, d' = diameter of grain, m

Table 1: Minimum Self-Cleansing Velocities

Pipe diameter (mm)	Minimum velocity (m/s)
150-250	0.7
300-400	0.8
450-500	0.9
600-800	1.0
900-1200	1.15
1300-1500	1.3
>1500	1.5

2.4 Groundwater Monitoring:

Groundwater embodies of different constituents at various concentrations. Monitoring of groundwater resource is as important as monitoring surface water resources. Basic resource monitoring to check the groundwater quality is done annually. This involves designing a sampling plan, selecting sampling stations, frequency and methods, bore drilling, decontamination and quality assurance. The bore drilling methods include rotary air drilling, rotary mud drilling, cable tool drilling, direct push technology, etc. which is followed by peizometric installations for bore construction. After construction of bores, samples are collected and analyzed as per Standard Methods. The analysis of groundwater in the study area yielded certain results, which are mentioned in Table 2.

Fig.A and **Fig.B** represents the ternary plots of major ions which are present in the study area. **Fig.C** represents graphically how the parameters varying in two separate seasons. These ternary plots help in identifying the sources of major ions in different samples. Atmospheric deposition forms the primary source of major ions, in which sodium and chloride are predominate This plot is done by converting the major ions to normalized concentrations and then to their respective percentages. The values of major ions obtained after the analysis are showed in the Table Among the total cations Na is the most dominant one followed by Ca, Mg and K. In anion ternary plot (Fig) all the samples are concentrated near bicarbonates and nitrate and sulphate. From the plot it is evident that the influence of chlorides is minimal. From the plots it is very clear that irrespective of the seasons the ion behave similarly. In the cation ternary plot (Fig. A and B), all samples have more lean towards Na+K. The large portion of Na may be due to atmospheric deposition, anthropogenic sources and rock weathering.

2.5 Methodology for Water Distribution System Design:

The design of water distribution network beginning from the supply of water from the overhead tank to every residential plot is carried out by using LOOP 3.0 version software. The network designed by the software is then simulated for increase demand conditions and reliability ascertained using increased peak factor in the designed network. The design of Under Ground Drainage System beginning from the sewage collection system to the septic tank is carried out by using K.U.W.S. and.D.B in-house excel spread sheet.

2.5.1 Proposed Scheme:

The existing layout is an isolated land deprived of any surface water source; hence the only reliable source is ground water and hence 3 bore-wells have been dug for the purpose. The water is pumped from these bore-wells to an Over Head Tank located at the highest level in the layout. From this the water is distributed to the entire layout through pipes under gravity.

2.5.2 Design of Components:

The present work aims at designing the components comprising bore-well, pumps, Overhead tank and distribution system. Design of components to meet the projected water requirements for the design period is to ensure a system that is reliable in hydraulic capability which ensures the stipulated minimum residual head. Design and stimulation of the distribution system are carried out with the aid of computer software LOOP 3.0 of UNDP. Data to be collected and processed or the design depends upon the input data required for the running of this software. The data required population, water demand, pipe line layout, pipe materials, pipe and work cost, hydraulic requirements and quality.

Table 2: Groundwater Analysis Report of Study Area

Parameters	Values Before Rainfall	Values After Rainfall
pH (mg/l)	7.51	7.49
Chlorides (mg/l)	7.2	7.18
Total Hardness (mg/l)	36	34
Calcium hardness (mg/l)	12	11
Magnesium Hardness (mg/l)	24	23
Alkalinity (mg/l)	20	19
Total solids (mg/l)	0.4	0.36
Total dissolved solids (mg/l)	40	37
Electrical conductivity (µS/cm)	62.5	62.3
Sulphates (mg/l)	8	7.88
Nitrates (mg/l)	1.96	1.93
Fluoride (mg/l)	1	0.89
Sodium content (mg/l)	58.1	58.09
Potassium (mg/l)	28.2	28.17
Dissolved oxygen (mg/l)	5	4.7

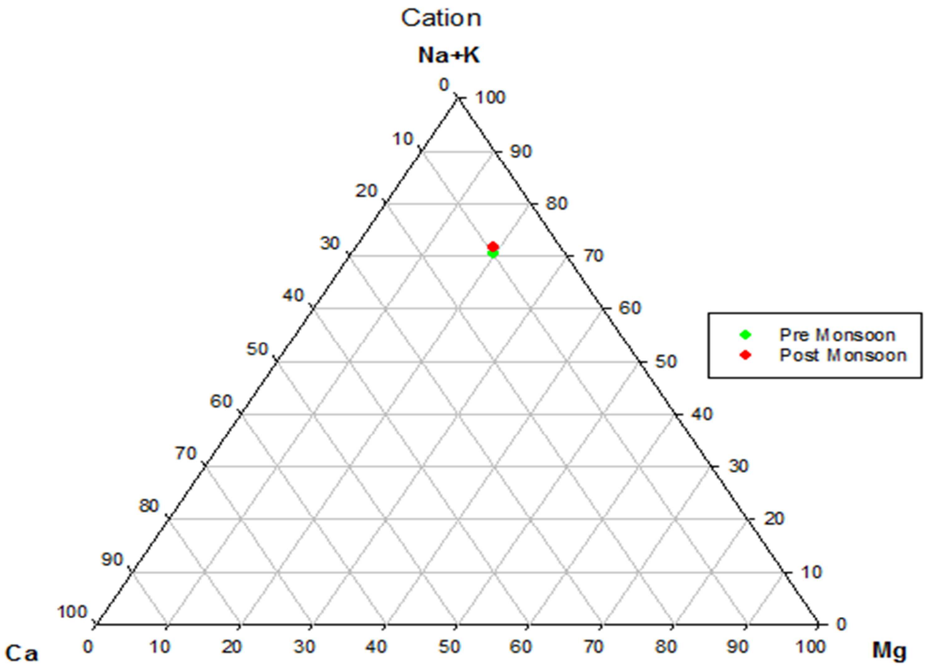

Fig A. Cation Concentration during Pre and Post Monsoon Season

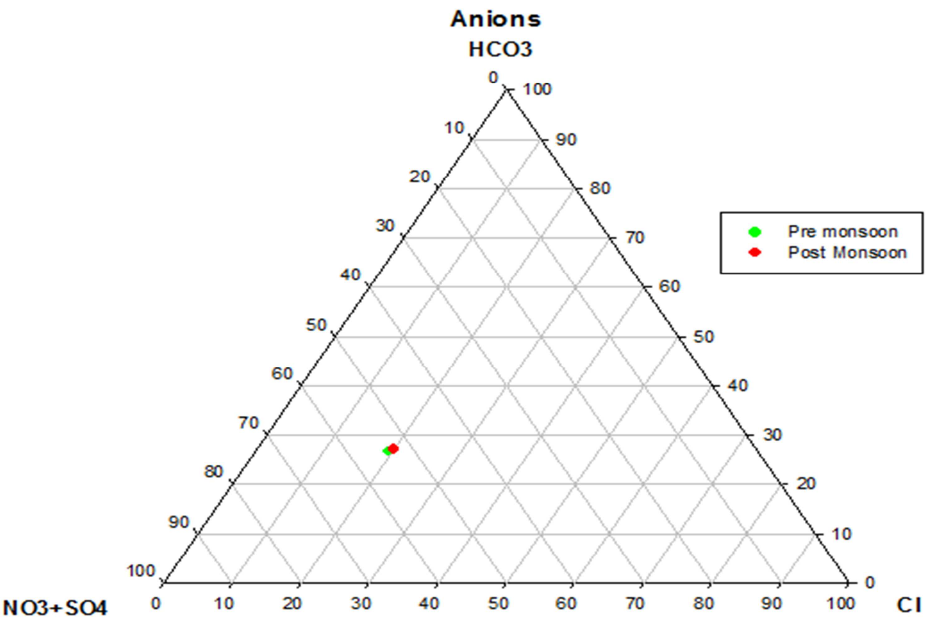

Fig B. Cation Concentration during Pre and Post Monsoon Season

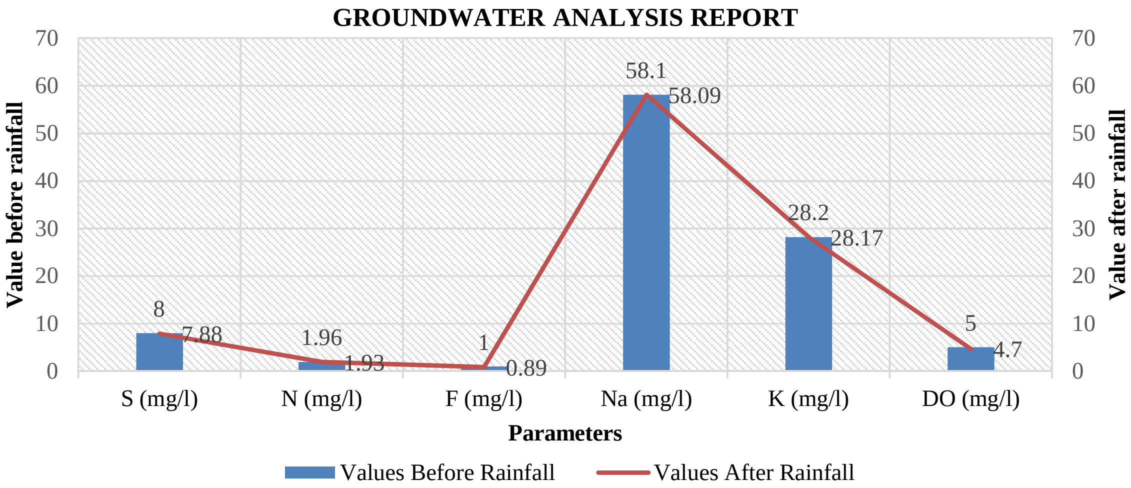

Fig C. Ground water analysis report of the study area for two seasons.

2.6 Design Data:

Various components of water requirements are designed based on the future population and geography of the region. The design of the scheme requires various data such as population details, site plan with length of roads and elevations, probable site for overhead tank, details for pipeline and other design criteria. Plan, reduced levels and other relevant data are collected from the Karnataka Urban Water Supply and Drainage Board, Mysore. Design criteria such as design period, water demand, peak factor are obtained from the Manual on Water Supply and Treatment by CPHEEO

2.6.1 Demands Pattern and Peak Factor:

The per capita rate of supply used in arriving at the total requirement of water is only the average demand, which may not be the actual flow rate of draw from the bore-well throughout the day. Rate of consumption varies with hour, day, month, and season in a year, demanding a design that is capable of ensuring supply rate at required terminal head during maximum rate of demand. A peak factor of 2.0 is therefore adopted in the design of pipe network. Peak factor is adequate according to the guidelines in the CPHEEO manual given in Table 3 (A.Raja et al., 2012)

Table 3: Peak Factor for Design

For population less than 50,000	3.0
For population 50,000 to 200,000	2.5
For population above 200,000	2.0

The demand patterns for any length of time, say 24 hours, describing the maximum demand with an average of 1 and maximum of 3.0 (Peak Factor) is adopted for analysis.

2.7 Input Data:

Design of pipe network is carried out with the computer aided model LOOP Version 3.0. LOOP program requires details of nodes, pipes, water demand, peak factor, reservoir data, details of pipe material, other fixtures, etc. Nodes data include information such as node number, peak factor, flow and elevation. Pipe data contains particulars of pipe number and length. Details of available types of pipes, diameter and class are to be furnished. Other features include reservoirs, pressure reducing valves and pumps. Cost of pipe per unit length including cost of material and labor charges is used for estimation of cost.

2.8 Mathematical Equations Used in the Analysis.

2.8.1 Pipe Head Loss Relationship

$$h_x = H_i - H_j = R_x Q_x^n \quad (3)$$

h_x= head loss in pipe X, H_i= HGL at upstream node H_j= HGL at downstream node, R_x= Pipe resistance coefficient, Q_x= discharge in pipe X, n= an exponent.

2.8.2 Node flow continuity Relationship

In steady state condition, algebraic sum of inflow and outflow at any node should be zero. The linear relationships are

$$\sum QX + qj = 0 \quad (4)$$

For j=1 to J, qj= External flow, j= number of nodes.

2.8.3 Lop head loss relationship

For all loops, algebraic sum of head losses of a loop is zero.

$$\sum\nolimits_c h_x = \sum\nolimits_c R_x Q_x^n = 0 \quad (5)$$

For c = I, II...NL, Where NL= number of loops.

2.9 Head Losses in Pipes:

Head loss occurs in pipes due to friction and minor appurtenances. Frictional head loss is the major head loss in pipe flow, which is determined by the following methods, in general.

2.9.1 Darcy Weisbach Equation:

The frictional head loss for incompressible flow is expressed in terms of density, velocity, and flow characteristics as,

$$h_f = \frac{(flv^2)}{2gd} \quad (6)$$

f= frictional coefficient, l= Length, v= velocity

2.9.2 Hazen Williams Formula:

$$V = 143.534 * C * R^{0.675} * S^{0.5525} \quad (7)$$

V = Velocity (m/s), C = Hazen William co-efficient
R = Hydraulic radius (m) S = slope of hydraulic gradient line.

2.10 Distribution System Design:

The heuristic design output provides a reasonable starting solution. Then the individual pipes are altered manually considering pressure requirements and flow conditions whereby the cost was brought down. The design is carried for a pressure requirement corresponding to a peak factor of 2.0 as per the guidelines of CPHEEO (MUD, 2014), LOOP 3.0 is a steady state design model that accommodates only one state of flow in one simulation. The pipe size which is sufficient to ensure the required discharge at demand nodes by satisfying the terminal pressure requirement is obtained from the model (Fernando 1996), which is further modified for cost reduction. The class of pipe selected also is in accordance with peak factor. But when water is available for more time during a day, the demand pattern varies according to the time-of-day and brings the peak factor to less than 1, may be 0.2 times the average demand which in turn causes the service reservoir to be full (Chambers et al. 2004). Consequently pressure in pipes increase at low demand times, which in turn requires use of a higher class of pipe at certain stretches where the allowable pressure is exceeded. This change of class of pipe to meet the pressure requirement has also to be made, selecting the appropriate size available and simulating again for hydraulic compatibility (Wood 2010).

2.10.1 Design of Service Reservoir for the Network

Intermediate design period	=15 years
Water demand	=285930 liters
Peak factor	=2.0
Average hourly demand	=11914 liters
Capacity of reservoir	=1, 50,000 liters

3. Methodology for Designing Under-Ground Drainage System:

A sewerage system can be viewed as a set of sewer lines collecting discharges at their nodal points and emptying into another set of sewer lines(Patil and Kulkarni 2014).Sustainable drainage is well known for its equal emphasis on water quality, water quantity, amenity and biodiversity (Charlesworth, 2010). Considering all aspects for introduction of a sewerage system, the shortest possible length of major sewers with branches and laterals coming from different pockets (DOP, 1995). There is increasing acknowledgment of the potentials of decentralized drainage system based on local treatment, attenuation, re-use, retention, and infiltration of precipitation runoffs(Roy et al. 2008)(Ashley et al. 2015)(Zhou et al. 2013). Also, attention was paid to ensure minimum depth of excavation with least number of sewage lifting stations. It is important to realize that all drainage systems are designed to a set of criteria that are subject to economic, social and environmental constraints(Kishore and Garg 2015). As per location, lifting stations were not required as the area follows a gentle uniform slope. The straight run between manholes shall be limited in length to 30 meters for sewers up to 300 mm diameter, and for sewers above 300 mm diameter the spacing of manhole may go up to 100m (UMA, 2011). One inspection chambers are provided for every three or four house where the building sewers are connected and later a single pipe connects to the main line which run across the road and thereby cutting of the road for each house.

3.1 Description of Computer Software for Design:

This software heuristically optimizes a sewer network for a given layout and rate of flow by minimizing the depth of excavation while meeting the design constraints such as minimum velocity, maximum velocity, maximum slope. The pipe size need not be given in the input data, because the program automatically selects the pipe size. For every pipe, a set of feasible diameters out of the set of specific commercially available diameters is identified subject to the condition that the velocity requirement and depth of flow constraints in the pipe are satisfied. Therefore this begins with the maximum permissible ratio of depth to diameter while working in the pipe slopes, tries to meet the constraint on progression of the pipe diameter, if specified and the design of the pipe slope which will be within the specific range of slopes.

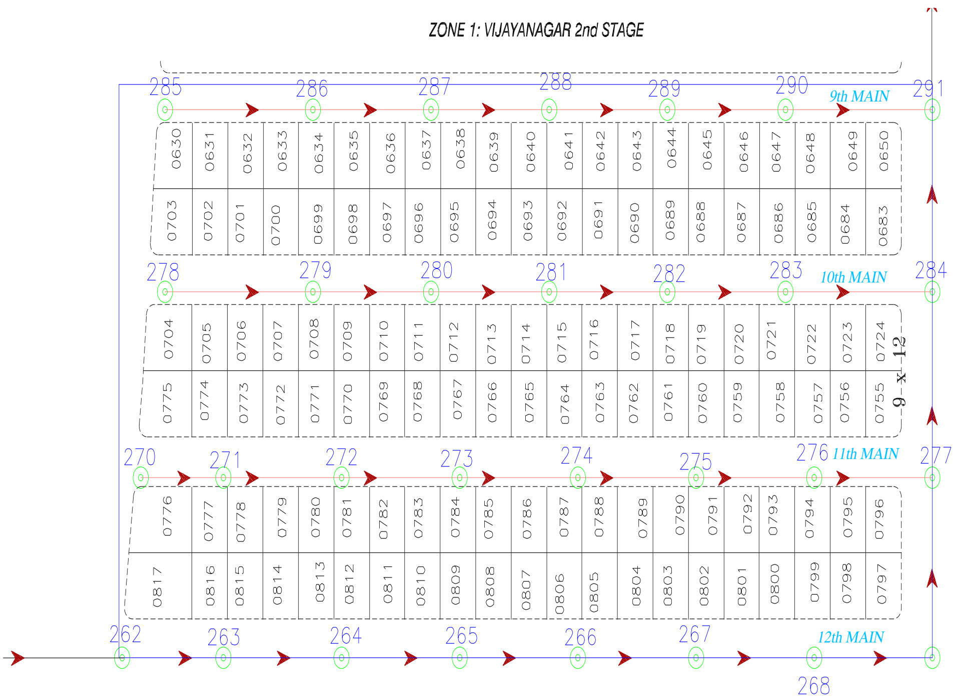

Fig D. Design of Sewer lines using Auto CAD 2007

3.2 System Design:

All node numbers were given at the starting point. Arrowheads were provided to indicate the direction of sewage flow Fig D. Contributory areas were judiciously taken and marked on the layout drawing for each and individual sewer stretch i.e. from one node to the next node of the sewer. These small areas were demarcated considering the possibility of laying branch and lateral sewer following the road network to serve the areas. Individual areas were then calculated and the populations to be served by these areas were computed by multiplying the average population density of the zone under consideration. The sanitary sewage being 80% of the per capita water consumption was estimated for each such contributory areas and groundwater infiltration (Treatment n.d.). The sum total of these figures gives the average flow of sewage. The sewers

are designed for peak flows and sewage treatment plants are designed for average flow (A. Raja et al. 2012; HEC, 2013 and (MPCA, 2002).
Lengths of each reach (sewer stretch) were measured in meters and design flow for each stretch was calculated in LPS.

$$Design\ flow = (Avg\ flow * peak\ factor) \quad (8)$$

The design velocities have been kept in the range of 0.8 to 3.0 m/s.

3.2.1 Mathematical equations used in the analysis.

3.2.1.1 Manning's equation

$$V = \left(\frac{k}{n}\right) R_h^{\frac{2}{3}} S^{\frac{1}{2}} \quad (9)$$

V = Mean Velocity (m/s), k = is a conversion factor 1 $m^{1/3}s^{-1}$ for SI, n = is the Gauckler–Manning coefficient, R = Hydraulic radius in m, S = Slope of Hydraulic Gradient Line.

3.2.1.2 Chezy's equation

$$V = C_h(RS)^{\frac{1}{2}} \quad (10)$$

C_h=Chezy coefficient, S= Slope of Hydraulic Gradient Line in m, R= Hydraulic radius in m.

4. Conclusion:

Based on the study following conclusions are made, The Use of software reduces manual calculations. Ground water quality which is taken as a source for the supply of water is good and within the standards. For fixing of pressure valves, scour valves and sluice valves, longitudinal sectional elevation profile should be considered that results in the safety of pipelines. For the depth up to 2 meters excavation the cost considered is 40 rupees, 2 to 4, 4 to 6 it is 45 and 52 respectively. Meanwhile for excavation from 6 meters to 7 meter depth the cost considered to be 55 rupees. For Zone 1 total number of pipes required to supply water are 65 in number. The total cost along with excavation cost and pipe cost is ₹ 834050. For Zone 2, 128 pipes are required with the total cost involving about ₹ 3698000. Zone 3 needs 54 pipes, the total pipe cost is ₹ 553130. The total cost along with excavation will come around ₹ 720270. Similarly for Zone 4, 5, 6 and 7 the total cost is ₹ 786420, ₹ 811480, ₹ 1109090 and ₹ 858590 respectively.

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References:

1) Afshar, M.H. (2005): Ninth International Water Technology Conference on 'Application of Max-Min Ant System For Joint Layout And Size Optimization Of Pipe Networks' Iran University of Science and Technology, 593–608.
2) Ali, M.E. (2015): Closure to "Knowledge-Based Optimization Model for Control Valve Locations in Water Distribution Networks" *J. Wat. Res. Pla. and Man.*, 141: 07015002.
3) Ashley, R. et al. (2015). Adaptable Addressing Occurrence Stormwater Urban Change and Uncertainty (AUDACIOUS) Drainage : in Intensity , 33(1): 70–84.
4) Babayan, A. et al. (2005): Least-Cost Design of Water Distribution Networks under Demand Uncertainty. *J. Wat. Res. Pla. and Man.*, 131(5): 375–382
5) Bong, C.H.J. (2014): A Review on the Self-Cleansing Design Criteria for Sewer System. *J. Civ. Engg.* 1–7.
6) Chambers K, Creasey J and Forbes L. (2004): Design and Operation of Distribution Network. World Health Organization. Safe Piped Water:Managing Microbial Water Quality in Piped Distribution Systems. IWA Publishing. London. UK 39-45.
7) A.Raja Jeya Chandra Bose, T.R.Neelakantan, P.Mariappan (2012): Peak Factor In The Design Of Water Distribution- Ananalysis, 3(2): 123-129.
8) De Corte, A. and Sörensen, K. (2013): Optimisation of Gravity-Fed Water Distribution Network Design: A Critical Review. *European J. Op. Res.*, 228(1):1–10.
9) HEC (2013) Hydraulic Engineering Circular : Urban Drainage Design Manual. HEC-22, 3rd Edition, Federal Highway Administration, USA.
10) UMA (2011): Town of Gibbons, Municipal Servicing Standards, UMA Engineering Ltd (2011).

11) Eusuff, M., Lansey, K. and Pasha, F. (2006): Shuffled Frog-Leaping Algorithm: A Memetic Meta-Heuristic For Discrete Optimization. *J. Engg. Opt.*, 38(2): 129–154.
12) Fernando, V. (1996): Water supply. (Energy and Environment Technology Source Books). London, UK: IT Publications.
13) Goulter, I.C. and Morgan, D.R. (2011): An Integrated Approach to the Layout and Design of Water Distribution Networks. *Civil Engineering Systems*, 2(2): 104–113.
14) MUB, Ministry of Urban Development (2014): Manual on Sewerage and Sewage Treatment Part B : Operation and Maintenance. (December).
15) Khudenko, B.M., Discussion of "Self-Cleansing Slope for Partially Full Sewers." *J. Envi. Engg.*, 113(3): 671–673.
16) Kishore, M.V. and Garg, A. (2015): Issues and Solutions of Drainage System for Dehradun City., 152–153.
17) Kumar, R., Singh, R.D. and Sharma, K.D., (2005): Water resources of India. , 89(5).
18) Lejano, R.P.(2006): Optimizing the Layout And Design of Branched Pipeline Water Distribution Systems. *Irrigation and Drainage Systems*, 20(1): 125–137.
19) Loureiro, D. et al., 2015. Water Distribution Systems Flow Monitoring And Anomalous Event Detection: A Practical Approach. *Urban Water Journal*,.1–11.
20) Chakrabarti, S. and Ashis Sarkar (2011): Pattern and Trend of Population Ageing in India. *The Indian Journal of Spatial Science,* 2(2): 1-11.
21) Patil, J.A. and Kulkarni, S.S. (2014): Design and Mapping of Underground Sewerage Network in GIS , a Case Study of Islampur Town. , 3(8): 424–431.
22) Pei-Te Chiueh, Shang-Lien Lo and Po-Yen Liao (1999): A Study of Self-Cleansing Velocity for Sanitary Sewer System Design, *Journal of the Chinese Institute of Environmental Engineering*, 9(4), 285-292.
23) MPCA, Minnesota Pollution Control Agency (2002): Design Flow and Loading Determination Guidelines for Wastewater Treatment Plants.
24) Quindry, G.E., Liebman, J.C. and Brill, E.D. (1981): Optimization of Looped Water Distribution Systems. *J. Env. Engg. Div.*, 107(4): 665–679.
25) Roy, A.H. et al. (2008): Impediments and Solutions To Sustainable, Watershed-Scale Urban Stormwater Management: Lessons from Australia and the United States. *J. Env. Man.*, 42(2): 344–359.
26) Charlesworth, S, M. (2010): A Review of the Adaptation and Mitigation of Global Climate Change Using Sustainable Drainage in Cities. *Journal of Water and Climate Change,* 1(3): 165-180.
27) Samani, H.M. V. and Mottaghi, A.: Optimization of Water Distribution Networks using Integer Linear Programming. *J. Hyd. Engg.*, 132(5): 501–509
28) Soltanjalili, M., Bozorg-Haddad, O. and Mariño, M. (2010): Effect of Breakage Level One in Design of Water Distribution Networks. *J. Wat. Res. Man.,* 25(1): 311–337.
29) DOP, Department of Planning, New South Wales (1995): Is an EIS required? : Best Practice Guidelines For Part 5 of the Environmental Planning and Assessment Act 1979.
30) Zhou, Q, Panduro, TE, Thorsen, BJ & Arnbjerg-Nielsen, K (2013): Adaption to Extreme Rainfall with Open Urban Drainage System: An Integrated Hydrological Cost-Benefit Analysis. Environmental Management, 51(3): 586-601.
31) Wood, D. (2010): KY Pipe Reference Manual. 1-76.