ANALYTICAL STUDY TO ASSESS AND EVALUATE METHODS OF ANALYSIS FOR WATER DISTRIBUTION METHODS

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ABSTRACT

In this study computer programs were developed to model water distribution network using developed computer routines, the following programs were employed: Hardy Cross Method, Finite Element Method, and Linear Theory Method. These computer techniques are studied and compared using two different head loss equations (i.e. Darcy Weisbach equation and Hazen William's equation). The procedure of finite element method that was used in this study, it was developed from the outline by Anthony Collins and Robert Jonson ref.(4); however it was modified in this study substantially to produce a workable method.

Due to the important role played by the calculation of the estimated values of the water demand at each junction in any water network, a technical routine to simplify the procedure for estimating the values of the consumptions (outflows) from each junction in skeleton network was presented in this study and compared with other available used procedures.
The results obtained from the computer analysis showed that the flow equation used had negligible effect on the flow distribution of flows in the network. However, it slightly affected the head-losses.

Six computer programs were written and successfully mounted using a realistic water distribution network. Benghazi water distribution network is used in this study as an application to run these computer programmers, the results of applying each of these six programs to analyze this network are presented and discussed in this study. Their success is shown by the close similarity of the results. Also, these computer results were verified by using a ready computer software package (WATERCAD V6.5) ref.(14).

From the results several recommendations are made to show the suitability of these methods.

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1 INTRODUCTION

In municipal water supply system, the transmission pipelines convey the collected water from the source to a water distribution network for the community. Since water distribution networks are analyzed by computer in this study, background knowledge about water distribution systems will be briefly present: The performance of a distribution water network can be judged on the basis of the head losses (pressure available) and the flow velocity for each pipe in the system. For normal supplies, water pressure at the street line must be at least 15m head to let water raise three stories and overcome the frictional resistance of the house connection and pipe work. Pipes should be able to carry flow with velocities that produce acceptable pressure drops. The minimum velocity should be 0.6 m/sec to prevent deposits and to maintain water quality. Velocity should be 1.2-1.8 m/sec to prevent problems of scour and not exceeding 2.5 m/sec. The hydraulic analysis of distribution network can only be approximate because of all factors affecting the flow cannot possibly be accounted for. In order to conduct the hydraulic analysis of a water distribution system, the patterns of the rates of water consumption should be estimated according to the technical specification of the water network distribution. The spatial distribution of consumption may be estimated by finding out population densities and public, fire fighting, commercial, and industrial use patterns known for the areas. For the purpose of design, analysis of a water distribution system is essential to determine the pressure and the flow. A full water distribution network solution consists of the calculation of all pipe flows and head losses. A network model is said to be balanced when the net flow into any junction is zero and when the net head loss around any loop is zero. Since the head loss in a pipe is a non-linear function of flow, iterative techniques are required.

2 THE OBJECTIVE OF THIS STUDY

Analysis of flow rates and pressures in pipe networks are needed whenever significant changes in pattern or magnitudes of demands are planned in municipal water distribution systems. These changes occur whenever new residential sub-divisions or industries attach to the existing system or new sources of supply are tapped. In the absence of such analysis to determine the performance of the existing system under the new demands accompanied by studies for providing the most economical alternatives for eliminating deficiencies, needlessly large investments are made for larger than necessary pipes, redundant lines or duplicate facilities. The purpose of this study is to gain the basic and fundamental concepts of water distribution network analysis, and to compare the effectiveness of the most commonly used methods of analysis for water distribution networks, using a real distribution system as an example. Six computer programs written in FRTRAN computer language will be build in this study using Finite Element Method; 'with using Darcy Weisbach equation (FEMD.F) and Hazen William's equation (FEMH.F)', Hardy Cross Method; 'with using Darcy Weisbach equation (HCMD.F) and Hazen William's equation (HCMH.F)', and Linear Theory Method; 'with using Darcy Weisbach equation (LTMD.F) and Hazen William's equation (LTMH.F)'.

2.1 Head Loss by Pipe Friction:

The head loss by friction is calculated in this study by using the Darcy Weisbach equation as following:

$$h_f = f * (L/D)*(V^2/(2*g)) \qquad (1)$$

Where:' f ' is a dimensionless friction factor, 'D' pipe diameter in m, 'L' is the length of the pipe in m, 'V' is the average velocity of flow in m/sec, 'g' is the acceleration due to gravity in m/sec^2.

2.2 Solution of the Darcy Equation by Computer:

Because of the equations for determining 'f' for smooth and transitional flow are complex and nonlinear it is usually necessary to apply a numerical method for calculation of the frictional head losses by the Darcy equation. One very effective method for obtaining 'f' in computer applications is to obtain an estimate of 'f' from the Von Karman equation as following:

$$1/\sqrt{f} = 1.14-2*\log_{10}(e/D) \qquad (2)$$

Where 'e' is the absolute roughness of the interior surface, and 'e/D' is the relative roughness. Initially assuming rough flow and then iteratively correcting this value of '*f*' by using the Colebrook and White formula, to calculate the friction factor:

$$1/\sqrt{f} = 1.14 - 2 * \log_{10}((e/D) + (9.35/(Re * \sqrt{f}))) \quad (3)$$

Symbols are as before.There are several ways of performance the iterations. Perhaps the best of these is the Newton-Raphson method which is explained below.

In Newton-Raphson method, the equation containing the unknown which we can call 'X' when describing the method in general is expressed as a function which equals zero when the correct solution is substituted into the equation F(X)=0. For instance the friction factor equation in transition region would be written as

$$F(f) = 1/\sqrt{f} - 1.14 + 2 * \log_{10}((e/D) + (9.35/(Re * \sqrt{f}))) = 0 \quad (4)$$

As one form of F(X)=0. In this method the unknown 'X' can be computed progressively by the following formula:-

$$X^{(m+1)} = (X^{(m)} - F(X^{(m)})) / (d F^{(m)}) / dX) \quad (5)$$

In which the superscripts in parenthesis are not exponents but denote number of iterations. A computer program for solving for 'f' for Darcy equation should include the following:

Reading in the specification such as D, e, V (or Q or Re), kinematic viscosity 'v', and L. If Reynolds number 'Re' is not a given specification then compute 'Re' and test whether Re < 2100, If so,

$$f = (64 / Re) \quad (6)$$

otherwise:

1. Compute the initial value for '*f*' from equation (2)
2. Compute $(e * V * \sqrt{(f/8)}/v)$ and if this quantity is greater than 100 then the ;*f*; from step 1 is correct, otherwise
3. Solve equation (4) by Newton method.
4. Calculation of the frictional head losses by Hazen-Williams Equation:

While Darcy equation is the most fundamentally reliable method for determining head losses in pipes, empirical equation are often used in practice. Perhaps the most widely used empirical equation is the Hazen-Williams equation, which is:

$$V = 0.85 * C * R^{0.63} * S^{0.54} \quad (7)$$

In which: 'V' is the velocity in m/sec, 'C' is the Hazen-Williams coefficient, 'R' is hydraulic radius in m, 'S' is slope of the energy line.

5. The head loss can be calculated by the following equation that was derived from equation (7) as following:

$$hf = 10.72 * L * Q^{1.852} / (C^{1.852} * D^{4.87}) \quad (8)$$

In which: Q is the flow discharge in m^3/sec., and 'hf' is the head loss in m.

3 DESCRIPITION OF METHODS ANALYSIS OF WATER NETWORK

In this investigation study three current digital techniques to balance distribution networks are reviewed. Two computer programs using FORTRAN 77 ref.(13) were written for each technique, the first one utilizes the Darcy-Weisbach equation for friction losses and the second one uses the Hazen-Williams equation.

3.1 Hardy Cross Method

This method was developed as a hand calculation method by Hardy Cross in 1936, see ref. (1). Is this method an initial flow assumption is required that satisfies flow continuity at all nodes. Then loop flow corrections are made one loop at a time until for each loop the algebraic sum of head losses around the loops approaches zero.

3.2 Solution Procedure

The procedure for solving a water distribution network using the Hardy Cross method may be summarized by the following steps:

1 Assume the most reasonable distribution of flows that satisfies the continuity equation at all junctions.

2 Select one pipe loop in the system and compute the net head loss for circuit based on the assumed flows:

$$\sum hf = \sum K_i * Q_i^{ni} = 0 \quad (9)$$

Where: 'K_i ' and' ni' are constants when dealing with single pipe of specific size and roughness. Subscript 'i' indicts the pipe number. Values for 'K_i ' and' ni' can be obtained directly from the Hazen-Williams equations as follows: $K_i = 10{,}72*L / (C_i^{1.852} * D_i^{4.87})$, and ni = 1.852; In which: C_i is the Hazen-Williams coefficient, if Darcy equation is being used, 'K_i ' and' ni' cannot be obtain directly. In this case, friction factor '*f*' must be approximated and 'K_i ' and' ni' should be determined.

3 Without regard to the sign calculate the sum of the' |hf/Q|' values.

4 Compute the flow correction using the following equation: $\Delta Q = - \sum hf / (n \sum|hf/Q|)$ and correct each of flows in the loop by this amount.

5 Apply this procedure to each pipe loop in the system repeating earlier circuits as necessary to obtain results within the desired accuracy.

3.3 Linear Theory Method:

In this method flow dependent linearised pipe resistances are iteratively corrected until a balance is achieved. Use of linear theory method requires a matrix equal in size the number of pipe, see ref. (1).

3.4 Solution Procedure

The procedure for solving a water distribution network using linear theory method may be summarized as follows:

1. (j-1) linear junction continuity equations are written.
2. L nonlinear energy equations are written around the loops of the distribution system.
3. K and n are obtained for the exponential formula.
4. The nonlinear energy equations are linearised by defining coefficients K' using equations are linearised by defining coefficients, K' using the following equation:

$$hf = [K_i * Q_{io}^{\,n-1}] * Q_i = K' Q_i \quad (10)$$

"In which: Q_{io} is the flow rate at the previous iteration, subscript 'i' indicates the pipe number". At the first iteration Q_o may be taken as unity for all pipes.

5. The N linear equations are then solved giving values of flow.
6. The values of K' are recalculated by using the new values of flow.
7. This process is repeated from stage 2 until convergence occurs.
8. Wood, and Carl, (ref. (3)), in developing the linear theory method suggest that for successive solutions after two iterations the average flow rate is computed as following:

$$Q_i^{(n)} = (Q_i^{(n-1)} + Q_i^{(n-2)}) / 2 \quad (11)$$

In which: $Q_i^{(n-1)}$ and $Q_i^{(n-2)}$ are the flow rates that are obtained from the previous two solutions for pipe 'i', $Q_i^{(n)}$ is the average flow rate for pipe 'i' which will be used for next iteration see ref. (1).

3.5 Finite Element Method for Water Distribution Network:

The method uses the relationship between the basic properties of each discrete element to define the behavior of that element. A solution for the response of the overall system subject to a set of boundary conditions is provided by solving a set of compatible simultaneous equations by matrix solution techniques.

3.6 Solution Procedure:

The solution used in this study for solving a water distribution network using finite element method was developed from that outlined by Anthony Collins and Robert Johnson; see ref.(4). However, it was modified substantially to produce a workable method. The modified procedure may be summarized as follows:

1. Select initial values of pipe coefficients C_i for each pipe using the following equation:

$$C_i = q_i / hf_i \qquad (12)$$

$$hf_i = K_i \; Q_i^n \qquad (13)$$

In which: 'hf_i 'is the head loss corresponding to 'q_i 'is calculated as explained before, C_i is the initial values of pipe coefficients for each pipe, q_i is the flow rate in the pipe it is convenient to choose 'C_i 'to correspond to the Reynolds number 'Re 'of 200,000 in each pipe, this being a typical value for a practical problem.' q_i' can be calculated as following:

$$q_i = 200{,}000 * v * A_i / D_i \qquad (14)$$

Then, these values are used to get a system of 'N' linear equations as following:

$$C_1 * hf_{i1} - C_2 * hf_{i2} = - QD \qquad (15)$$

2. The system matrix is then solved for the value of the head loss in each pipe.

The flow in each pipe is calculated again by using the initial value of pipe coefficient 'C_i 'and new value of head loss to get the new values of flow. Substituting these new values of flow into equation (13) gives new values of head loss.

3. The third and final step required is to change the value of 'C_i' as follows:

$$C_1 \text{ 'new'} = q_i \text{ 'new'} / hf_i \text{ 'new'} \qquad (16)$$

4. These steps are repeated until convergence occurs, see ref. (12).

5. APPLICATION OF THE COMPUTER PROGRAMS TO ANALYZE THE WATER DISTRIBUTION NETWORK FOR BENGHAZI

For purpose of comparison there digital techniques are used in analyzing the water distribution network for Benghazi to determine the flow rates and head losses in each pipe. Benghazi city is the second largest commercial city in Libya, located to the north-east part of Libya with population of 1,037,000 people in year 2014 (ref. (5)).

5.1 The Water Supply System for Benghazi:

The main source of water which feed the city is the manmade river project through Talhia distribution reservoir. Junctions 1, 3, and 9 are input junctions for the Benghazi water distribution network ref. (11) .Figure 1 shows the skeleton network of the Benghazi water distribution.

5.2 Specifications for Design Parameters:

The water distribution network for Benghazi has been designed by a consulting firm under the responsibility of the municipality of Benghazi. It is designed up to the year 2014. The following design criteria were assumed by the firm ref. (5):

The average water consumption is 270L/person/day in year 2014. The maximum hourly water consumption is 250-300% of the average consumption. The maximum daily water consumption is 160-180% of the average consumption. The velocity of fluid varies between 0.6-1.3 m/sec. The hydraulic gradient in the main pipes varies between 0.1-0.3 percent. An additional 20 L/sec flow capacity is required in all pipes for firefighting ref. (5).

5.3 Necessary Data for Application:

The water distribution network for Benghazi was simplified by ignoring most pipes of less than 400 mm diameter. This left a skeleton network consisting of 23 individual loops, 65 pipes and 43 nodes (junctions) as shown in Figure 1. We assumed that all consumption in the skeleton network was taken off at the nodes. Where two or more pipes run in parallel between nodes, they were replaced by a single pipe of equivalent diameter. We estimated nodal demands based upon population patterns of the city. We used the population of 1,037,037 inhabitants for the year 2014, to get the rate of water demand. The total demand in each loop was estimated accordingly using the master plan for city of Benghazi in year 2014 ref. (5, and 11). This estimated total demand in each loop was distributed between the junctions around its junctions. One of the most commonly used ways is to distribute such demand between the surrounding junctions in each loop according to the estimated amount of demand on each pipe line around the loop. In this method the area inside the loop will be divided between the surrounding pipes by using geometric ways, so each junction's demand in the loop was estimated using a percentage value from the total demand the value of this estimated percentage depend on the contribution area in front of each pipe. In this study an alternative procedure was proposed to simplify this procedure. Instead of calculating the contribution area in front of each pipe the length of each pipe line is used directly, this way will simplify and improve the procedure to make it much easier to write a computer program to calculate such work especially when the shape of the loop is complicated.

Figure 1 shows the junction demand. The rate of water demand was used in determining the flow rates head loss in each pipe. The configuration of water distribution network and assumed nodal takeoffs are shown in Figure 1 for a total consumption 2984.3408 L/sec. We assumed that the viscosity of water and the roughness of the pipes are constant and numerical values of them are $1.131*10^{-6}$ m^2/sec and $2.6*10^{-4}$ m respectively. We also assumed that Hazen-Williams coefficient is constant and is equal to 130.

5.4 Running the Programs:

Microsoft Developer Studio FORTRAN Power Station 4.0 package software ref. (13) was used in this work to build and run these developed computer programs. Three files are required to run these programs, one is of data information about the pipes (length, diameter, initial pipe flow, and junction demand) in the network, and the other for storing parts of the results in a form useable by Microsoft excel software. An interactive program is written in order to help the user to create the data files for each method containing the information about the network by asking questions that appear at the user's computer screen.

6 RESULTS AND CONCLUSIONS

Computer programs have been written for three different methods (i.e. Hardy Cross, Linear theory, and Finite element methods) using two flow formulae. The main purpose of this study is to compare the effectiveness of the different methods of analysis, including the results of the application area that was used in this study is included in this discussion. Due to the large amount of result data, summarized results were placed in figures to give a quick view for the purpose of comparison; for verification of my computer results WATERCAD software package ref. (14) is used and its result is included in this comparison study.

6.1 Interpretation of Results

Note that in the computer results the flows for pipes numbers 4, 18, 20, 24, 46, 49, 54, 58, and 63 have negative signs. This means that the direction of the flow in these pipes is opposite to the initially assumed directions of flow. Note also that the error value is equal to the summation of the difference between the flows for each pipe in the system from the past two iterations divided by the number of the pipes in the system.

6.2 General Notes

It is noted that all the computer programs that used the same head loss equation gave identical results for analysis of the network, although the number of iterations varied considerably as did the computing time. Slight differences occur when comparing the programs that used differences head loss equations, which are negligible. The maximum difference between the results of Hazen Williams and the result of Darcy equation are $4.334*10^{-3}$ m^3/sec, $4.333*10^{-3}$ m^3/sec, and $4.335*10^{-3}$ m^3/sec when using Finite element, Hardy Cross, and Linear theory methods respectively. Also, for verification the computer results the ready software WATERCAD ref. (14) is used in this study and its results shows a slight differences comparing the program's results, the average maximum difference is not more than $1.63*10^{-3}$ m^3/sec, the consistency of the results and the results of WATERCAD software shows that there are unlikely to be major errors in the programs .Figure 2 shows the comparing the results of the calculated flow in each pipeline in (m^3/sec) using, Finite Element Method (FEMD.F, and FEMH.F), Hardy Cross Method (HCMD.F, and HCMH.F), and Linear Theory Method (LTMH.F, LTMD.F), and WATERCAD Ready Software. Also, Table 1 shows the comparing the Results of the calculated pressure head at each Junction in the Benghazi water distribution network (Phj) in meter (m) by using all the methods that mentioned above. The comparison indicates the close similarity of the results.

Figure 3 shows the comparing the results of the calculated velocity of flow in each pipe in (m/sec) using the average water demand in year 2014. From the results, the maximum and minimum values of the velocity are 2.393 m/sec and 0.007 m/sec respectively. More than 55% of the values of velocities in the Figure lay between 0.6 m/sec and 2.5 m/sec which match the specifications on which the distribution network for Benghazi has been designed. Also, Figure 3 shows that 35% of the pipes in this water distribution network system have velocities less than 0.6 m/sec. some of these very low velocities may be caused by uncompleted planning of some areas in the master plane of the city of Benghazi such as Jalyana area.

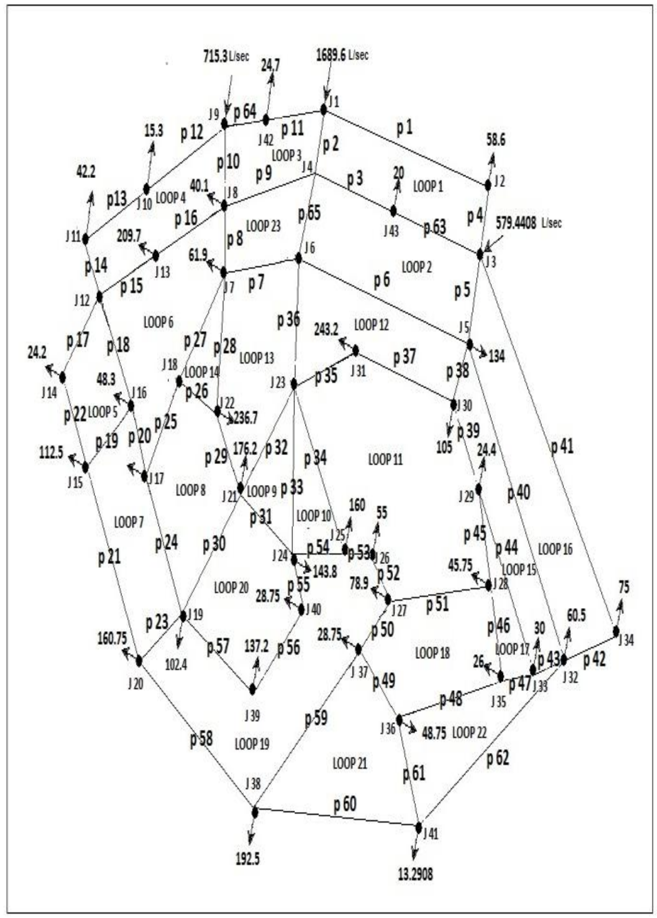

Figure 1. Configuration of water distribution network and assumed nodal takeoff and direction of flow for water demand for City of Benghazi (year 2014)

Table 1: Comparing the Results of the calculated Pressure Head at Junction (Phj) in meter (m) using, Finite Element Method (FEMD.F, and FEMH.F), Hardy Cross Method (HCMD.F, and HCMH.F), and Linear Theory Method (LTMH.F, LTMD.F), and WATERCAD Ready Software

J NO.	P-HEADJ (M) USING FEMH.F	P-HEADJ (M) USING FEMD.F	P-HEADJ (M) USING LTMH.F	P-HEADJ (M) USING LTMD.F	P-HEADJ (M) USING HCMH.F	P-HEADJ (M) USING HCMD.F	P-HEADJ (M) USING WATERCAD
1	30.0000	30.0000	30.0000	30.0000	30.0000	30.0000	30.0206
2	28.2503	28.2866	28.2503	28.2865	28.2540	28.2900	27.9739
3	30.0000	30.0000	30.0000	30.0000	30.0000	30.0000	30.8930
4	27.1004	27.0022	27.1004	27.0022	27.1005	27.0023	27.0924
5	22.4954	22.2487	22.4954	22.2487	22.4966	22.2500	23.5846
6	31.2336	31.0497	31.2336	31.0497	31.2337	31.0498	31.2154
7	31.9870	31.8727	31.9870	31.8728	31.9863	31.8722	31.9770
8	30.7216	30.6117	30.7216	30.6117	30.7211	30.6113	30.7163
9	30.0000	30.0000	30.0000	30.0000	30.0000	30.0000	32.4009
10	35.3599	35.4042	35.3599	35.4042	35.3592	35.4036	37.7612
11	34.6531	34.2943	34.6531	34.2943	34.6465	34.2876	37.0468
12	30.2741	29.9464	30.2741	29.9463	30.2672	29.9394	32.6663
13	23.2746	22.9462	23.2746	22.9462	23.2692	22.9408	25.6671
14	32.9218	32.6172	32.9218	32.6172	32.9145	32.6099	35.3128
15	31.5682	31.2953	31.5682	31.2953	31.5603	31.2875	33.9564
16	32.6904	32.4046	32.6904	32.4046	32.6872	32.4014	32.6813
17	33.1604	32.8872	33.1604	32.8872	33.1571	32.8839	33.1490
18	28.8587	28.5629	28.8587	28.5629	28.8559	28.5602	28.8445
19	32.2140	31.9562	32.2140	31.9562	32.2110	31.9533	32.2005
20	33.9784	33.7442	33.9784	33.7442	33.9767	33.7424	33.8417
21	22.5712	22.2854	22.5712	22.2854	22.5682	22.2824	22.5534
22	25.5749	25.2890	25.5749	25.2890	25.5719	25.2861	25.5581
23	27.5473	27.3047	27.5473	27.3047	27.5476	27.3051	27.5215
24	24.5685	24.2842	24.5686	24.2842	24.5701	24.2858	24.5502
25	24.6101	24.3241	24.6101	24.3241	24.6121	24.3261	24.5904
26	24.2824	24.0302	24.2824	24.0302	24.2842	24.0320	24.2566
27	26.0244	25.7881	26.0244	25.7881	26.0261	25.7898	25.9909
28	28.5807	28.3203	28.5807	28.3203	28.5835	28.3231	29.7292
29	26.9423	26.6910	26.9423	26.6910	26.9440	26.6929	28.0697
30	24.5271	24.2779	24.5271	24.2779	24.5285	24.2794	25.6511
31	26.8305	26.6309	26.8305	26.6309	26.8313	26.6316	26.7645
32	28.2598	28.0366	28.2598	28.0366	28.2623	28.0391	29.3552
33	26.5922	26.3602	26.5922	26.3602	26.5943	26.3624	27.7118
34	32.1008	32.1877	32.1008	32.1877	32.1013	32.1882	33.0413
35	27.5842	27.3254	27.5842	27.3254	27.5871	27.3283	28.7316
36	26.2612	26.0131	26.2612	26.0131	26.2654	26.0172	27.4360
37	24.4645	24.2464	24.4645	24.2464	24.4662	24.2480	24.4257
38	30.8589	30.6467	30.8589	30.6467	30.8620	30.6495	30.8414
39	31.9720	31.7330	31.9720	31.7330	31.9690	31.7300	31.9596
40	24.1506	23.8726	24.1506	23.8726	24.1522	23.8742	24.1338
41	31.1894	30.9497	31.1894	30.9497	31.1945	30.9547	32.3634
42	31.3990	31.4279	31.3990	31.4279	31.3984	31.4274	31.4011
43	28.4633	28.4510	28.4633	28.4511	28.4644	28.4520	28.2673

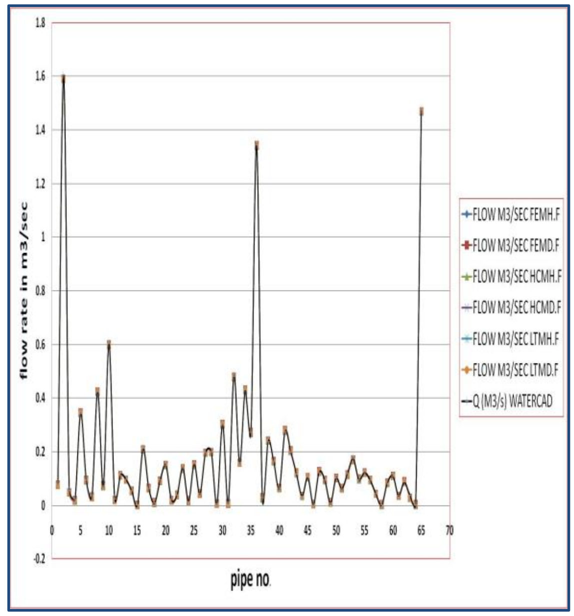

Figure 2: Comparing the Results of the calculated flow in each pipe in (m^3/sec) using, Finite Element Method (FEMD.F, and FEMH.F), Hardy Cross Method (HCMD.F, and HCMH.F), and Linear Theory Method (LTMH.F, LTMD.F), and WATERCAD Ready Software

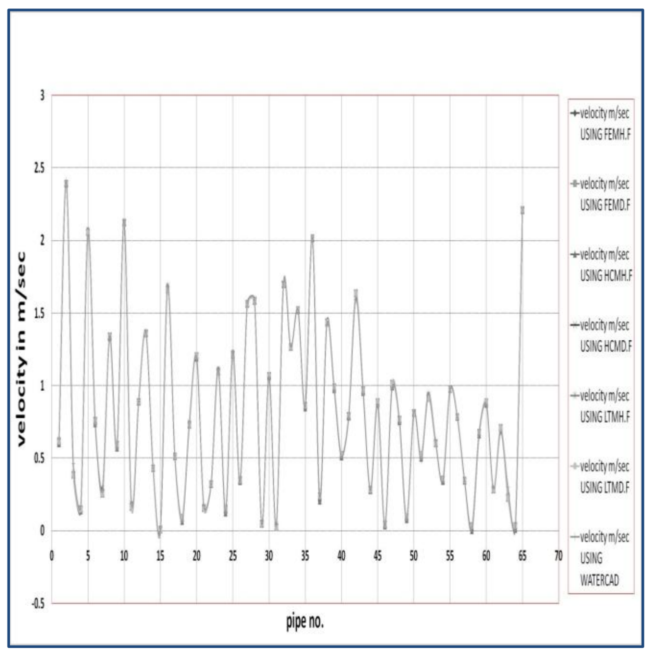

Figure 3: Comparing the Results of the calculated the velocity of flow in each pipe in (m/sec) using, Finite Element Method (FEMD.F, and FEMH.F), Hardy Cross Method (HCMD.F, and HCMH.F), and Linear Theory Method (LTMH.F, LTMD.F), and WATERCAD Ready Software

6.3 Comparison of the Current Three Methods:

a) Hardy Cross method requires an initial set of feasible flows. For rapid convergence this set of assumed flows should be the most reasonable distribution of flows that satisfies the continuity equation at all junctions and close to the true solution as possible.
b) Hardy Cross method requires less computer storage than other two methods, and it is easier to write a computer program for this method rather than the other methods. But it requires more iterations than the other methods. The numbers of iterations at error value equal to 0.00001 for Hardy cross, Finite Element, and Linear Theory methods are 38, 7, and 7 respectively.
c) In comparison with linear theory method or Finite element method, it requires less computer storage not only because the number of simultaneous equations is limited to the number of loops, but also because it requires less data storage for a given size of network.
d) The linear theory method does not require an initial feasible flow assumption to get solution.
e) When we examine the Finite element method we note that it requires selecting an initial value of flow which can be calculated by assuming the value of Reynolds number for each pipe to estimate the initial assumption of flow for each pipe. It is suitable to use Finite element method to solve an unlimited network size.

7 CONCLUSIONS

a) Three methods using two equations were used. All six programs were successfully mounted and tested using a realistic network. Their success is shown by the close similarity of the results. Also, these computer results were verified by using a ready computer software package (WATERCAD V6.5).
b) The validity of the results for the Benghazi network is limited due to lack of detailed information concerning distribution of demands. This, however, is not important as the main purpose was to test the methods of analysis.
c) The flow equation used had negligible effect on the distribution of flows in the network. However, it slightly affected the head losses.
d) The hardy Cross method is quicker than other two methods. No difficulty in convergence was experienced in this study using the Hardy Cross method.
e) There seems, on balance, little to chose between the methods, However, for use on a micro-computer, the Hardy Cross method is preferable, requiring a shorter programs and less computer storage.
f) In this study an alternative procedure to estimate the junction's demand in the loop was proposed to simplify this producer. Instead of calculating the contribution area in front of each pipe the length of each pipe line is used directly, this way will simplify and improve the procedure, and to make it much easier to write a computer program to calculate such work especially when the shape of the loop is complicated
g) The all computer programs showed a more flexibility and control to deal, with the required data values that required for each inflow junction especially when we enter the fixed remaining pressure head into the inflow junction (steady state condition), than the ready computer software package (WATERCAD V6.5); see table 1.

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