WATER PRESSURE EQUALIZATION IN PIPE NETWORK CASE STUDY: AL-KARADA AREAS IN BAGHDAD

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ABSTRACT
In order to make a balance between the increasing of potable water demand and the available quantity, a pipe network should be managed in an optimal hydraulic operation state. The optimal operation of a water supply network leads to minimize the effect of the variance in pressure between the available and minimum required pressure head. It simulates the hydraulic model and puts the optimized project with the constraints (minimum design head=20 m, and the available commercial pipe diameter, $1600 \geq D \geq 250$ mm). The objective function is to minimize the cost for the suggested hydraulic solution to a minimum value. Pressure uniformity coefficient (UC), Standard deviation (σ) and coefficient of variance (Cv) are used to show that the pressure head at the nodes of the network is uniformly distributed. The optimal design of the case study (R9 water supply network) has an actual cost of 561,169,310 ID and the uniformity indices of UC=99.565, σ=3.6508 and Cv=0.1543 while the existing design has cost of 856,617,170 ID with the uniformity indices of UC=97.909, σ=3.5977 and Cv=0.7906. Hence there is a benefit of 34.5% in the cost of the optimal design used in this study, with high uniformity coefficient. The effect of Hazen-William coefficient (C) on total cost showed an inversely linear effect. For the value of C=130, the actual cost was 600,898,300 ID, i.e., the penalty cost approached to zero and has no effect on the total cost.

الخلاصة
لغرض عمل موازنة بين ازدياد الاحتياج والكمية المتوفرة للمتطلب المائي وجب إدارة شبكة المياه بالشكل الأمثل للأداء الهيدروليكي.إن أفضل و أمثل ادارة هيدروليكية لشبكات المياه تقود إلى تقليل الفارق بين الضغط المتوفر من قبل الشبكة واقل ضغط مطلوب بالشبكة إلى اقل حد ممكن. عمل محاكاة إلى النموذج الهيدروليكي ووضع المشروع الأمثل مع المحددات وهي (اقل ضغط تصميمي هو 20 m و الأقطار التجارية المتوفرة $1600 \geq D \geq 250$). إن دالة الهدف تعمل على جعل كلفة الحل المقترح إلى اقل قيمة ممكنة. معامل انتظام الضغط UC, الانحراف المعياري σ والتباين Cv تستخدم كمؤشر إلى أن الضغط بالعقد للحل الأمثل للمشكلة يمتلك انتظاماً توزيعياً. أفضل تصميم لشبكة الدراسة الحقلية (شبكة ماء الكرادة R9) يمتلك كلفة هي 561,169,310 ID ومعامل الانتظام UC=99.565 , الانحراف المعياري σ=3.6508 والتباين Cv=0.1543 , بينما التصميم الحالي للشبكة بكلفة 856,617,170 ID ومعامل الانتظام UC=97.909 , الانحراف المعياري σ=3.5977 والتباين Cv=0.7906 و بذلك التصميم الأمثل في هذه الدراسة اقل كلفة بنسبة 34.5% من التصميم الحالي وكل معاملات الانتظام تدل على إن

الشبكة ذات توازن وانتظام عالي. إن تأثير معامل الاحتكاك Hazen-William على الكلفة الكلية يظهر تأثيراً عكسياً, ولقيمة C=130 تكون الكلفة الكلية وهي ID 600,898,300 . بذلك تكون ال(Penalty cost) صفراً ولايوجد لها تأثيراً على الكلفة الكلية.

KEY WORDS:
Water pressure, pressure equalization, Al-Karada water supply, pipe network, water supply, optimal design, pressure uniformity distribution

INTRODUCTION

Water is one of the essential elements of life, from early days men soon realized that rivers and streams in their natural states seldom provide, adequate water to satisfy their needs. Water is vital for human existence; without water there is no life on earth (Anis et.al., 1977) .
A water distribution network is a system containing pipes, reservoirs, pumps, valves of different types, which are connected to each other to provide water to consumers. It is a vital component of the urban infrastructure and requires significant investment(Abeb and Solomatine, 2000). The analysis of water distribution network means evaluation of quantity of water flowing through each pipe and pressure head at junction (node) of the system, while the design of water distribution network means evaluation of the diameter of each pipe and the optimum configuration according to specified requirements(Don, 1981).
The problem of optimal design of water distribution network has various aspects to be considered such as hydraulics, reliability, material availability, water quality, infrastructure and demand patterns. Even though each of these factors has its own part in the planning, design and management of the system and despite their inherent dependence, it is difficult to carry out the overall analysis. Previous research indicates that the formulation of the problem on a component basis is worthy doing. In the present study, the problem is posed as a global optimization. The optimization model determines whether the design is optimal or not, if not, the optimization model based on reducing the variance between the minimum required and modeled pressure head at the nodes. This paper deals with the determination of the optimal diameters of pipes in a network with a predetermined layout. This includes providing the pressure and quantity of water required at each demand node. An appropriate interface is created between a global optimization tool with the various random algorithms, and a network simulation model that can handle steady state condition.

EXTENT OF THE PROBLEM

The problem reduced to such an extent has two constraints from hydraulic requirements. The continuity constraints, states that the discharge into each node must be equal to that leaving the node, except for storage node (tanks and reservoirs). This secure, the overall mass balance in the network. For n nodes in the network, this constraints can be written as:

$$\sum_{i=1}^{n} Q_i = 0 \tag{1}$$

where Q_i represents the discharges into or out of the node i (sign included).
The second hydraulic constraint is the energy constraint according to which the total head loss around any loop must add to zero or is equal to the energy delivered by a pump if there is any:

$$\sum hf = Ep \tag{2}$$

where hf is the head loss due to friction in a pipe and Ep is the energy supplied by a pump. This embeds the fact that the head loss in any pipe which is a function of its diameter, length and hydraulic properties must be equal to the difference in the nodal heads. This constraint makes the problem highly non-linear owing to the nature of the equation that relates frictional head loss and flow. This equation can be written as:

$$hf = \frac{aQ^b}{D^c} \tag{3}$$

Where a is coefficient depending on length, roughness coefficient of the pipe, b is discharge exponent and c is exponent of pipe diameter (D) which is very close to 5 in most head loss equations(Abeb and Solomatine, 2000).

Considering the diameter of the pipes in the network as decision variables, the problem can be considered as a parameter optimization problem with dimension equal to the number of pipes in the network. Market constraints, however, dictate the use of commercially available (discrete) pipe diameters. With this constraint the problem can be formulated as a combinatorial optimization problem (Abeb and Solomatine 2000).

The minimum head requirement at the demand node is taken as a constraint for the choice of pipe diameter. Even though the use of an exhaustive search guarantees finding the global optimum, the fact that the computational time increases expontially with the dimension of the problem makes it impractical to apply them in a multimodal function like this, and especially for real life-size problems.

REVIEW OF PREVIOUS RESEARCH

Various researches have addressed this problem in a number of different ways during the past decodes. Thawat (1973), produced a non-linear programming model for computing the pipe sizes and pumping capacities that minimize the total cost, to satisfy the demand requirements. Pramod (1979), determined a method based on the critical path concept to select the optimal sets of pipe sizes for optimization of branch network by linear programming. Gerald et.al. (1981), described a gradient technique for optimization of pipe networks. It is possible to use the value of node head, the pipe flow, to calculate the gradient term ∂(cost)/∂(Hi) for each node in the network. Pramod (1983), developed a method for optimal design of multi source, looped, gravity- fed water distribution systems subjected to a single loading pattern. The method is based on linear programming technique and produces a locally optimal solution. Ronald and Karime (1983), proposed a method for least cost design of water distribution network which is based on a traditional technique of pipe network analysis. Cenedes et.al. in (1987), determined an optimal design and operation of closed hydraulic network with pumping stations and different flow rate conditions. Yu-chun et.al. (1987), utilized a model that can be used to determine the least cost design of water distribution system subjected to continuity, conservation of energy nodal heads, and reliability constraints. Kevin and Lary in (1989), determined the optimal settings for controls and pressure reducing valves. This methodology couples was based on non-linear programming technique. Ian and Fracols (1990), established a new methodology for reliability considerations directly into least cost optimization design and operation models for water supply networks. Nowar and Abbas, (1997), presented a linear programming gradient model with mathematical corrections to find the optimum (least cost) design to pipe networks for constant and variable pumping head. Objective function to be minimized represents the overall cost of the pipelines and cost of the pumping station in the case of variable pumping head. Bogumil et.al (1998), demonstrated optimization analysis by solving inverse problem such as optimal scheduling, model calibration and design. A new generic optimization approach based on a continuous assumption and the use of non-linear mathematical programming is proposed. Abebe and Solomatine (2000), presented an approach to the optimal design of pipe networks for water distribution. The problem was solved using a global optimization tool with various random search algorithms and dynamic loading conditions. The proposed optimization setup can handle any type of loading condition and neither makes any restriction on the type of hydraulic components in the network nor does it need analytical cost functions for the pipe. Paul et.al (2002), established a new management model for optimal control and operation of water distribution systems. The proposed model makes use of the latest advances in genetic algorithm optimization to automatically determine the least cost pump scheduling operation policy for each pump station in the water distribution system while satisfying target hydraulic performance requirements.

PROBLEM FORMULATION

Constraint handling

The constraint in the problem can be grouped into the following: hydrodynamic, minimum head and commercial constraints. The hydrodynamic constraints are handled by the function network simulation model. The optimization function handled the upper and lower bound on parameter, while penalty function was used to handled minimum nodal constraints. Commercial constraints reduce the parameter space to a discrete one.
This can be adjusted to the number of available commercial pipe sizes, therefore, the search algorithms will be for the optimal pipe diameters.

Objective function

The objective function to be minimized by the optimization algorithms is the cost of the network. If the actual cost of the network is the sole objective function, then obviously the search will end up with the minimum possible diameters allocated to each of the pipes in the network. To tackle this, a penalty cost is added to the actual cost of the network based on the minimum head constraint.

Actual cost of the network

The actual cost of the network (Ca) is calculated based on the cost per unit length associated with the diameter and the length of the pipes

$$Ca = \sum_{i=1}^{n} C(D_i)\, L_i \tag{4}$$

Where n is the number of pipes in the network and $C(D_i)$ is cost per unit length of the i^{th} pipe with diameter (D_i and length L_i.

Penalty cost

The penalty cost is superimposed on top of the actual cost of the network in such a way that it will discourage the search in the infeasible direction. It is defined on the basis of the difference between the minimum required pressure head (H_{req}) at the node and the lowest design pressure head obtained after simulation. It depends upon the degree of pressure violation and the cost of the network in some cases and is defined in the following way:

1- For networks in which all the nodal heads are greater than (H_{req}) the penalty cost is zero.
2- for the networks in which the minimum head is greater than zero but less than (H_{req}), it increases linearly with the nodal head deficit, i.e.:

$$C_{penalty} = P * \sum_{j=1}^{nn} (H_{\text{mod.}} - H_{req.}) * CT \tag{5}$$

Where P is a penalty coefficient and CT is the possible cost for each variance between modeled pressure head (after simulation) and minimum required head (calculation on the cost of the commercial pipe available) in (ID/1m), CT is a function of the pipe diameter cost, i.e.:

$$CT = A D^{B} \tag{6}$$

Where A,B are a constant depending on the available commercial pipes, D is the pipe diameter in (mm) and H_{req} is the minimum required pressure head (m).
Hence the total cost (C_{total}) will be:

$$C_{total} = Ca + C_{penalty} \tag{7}$$

<u>Optimal design of water supply network</u>

The method used in this paper is suited for engineers of less experience in the design of water networks. The design procedure is performed by a computer program in the following manner:
After assuming the initial values of the diameter the network is analyzed by using the Hardy-Cross method (Quantity balance method). The pipes will be arranged according to the hydraulic gradient in a decreasing form, to find the lowest pressure in the network. If the modeled pressure (calculated after analysis) is less than the designed pressure then the pipe diameter of the high hydraulic gradient is increased, using larger commercial available diameter. After changing the pipe's diameter, the analysis will be repeated to finish the first attempt. The program will repeat this process until the lowest pressure reaches the designed pressure head.
For pipes of low hydraulic gradient the program will choose a smaller commercial diameter to ensure pressure equalization. The program will repeat maximization and minimization for the pipe diameter until the optimal design is reached, then the network cost is to be calculated to give the minimum cost.

INDICATORS OF THE UNIFORMITY OF THE NETWORK

To identify the uniformity of the pressure distribution in the network, the following indicators are used.

<u>Standard Deviation (σ) and Coefficient of Variance (Cv)</u>

Considering the Standard deviation (σ) an indicator to distribute the data from the arithmetic mean.The data used to calculate the Standard deviation (σ) are the modeled pressure heads.

$$\sigma = \sqrt{\frac{\sum_{i=1}^{nn} (P_i - P_m)^2}{n}} \tag{8}$$

Where P_i: Pressure head at node i in (m), P_m: Arithmetic mean (average pressure value in the nodes) in (m) and n total no. of nodes.
The coefficient of variance (Cv) is formed by:

$$Cv = \frac{\sigma}{P_m} \tag{9}$$

Whenever the value of the coefficient of variance (Cv) is near zero, this indicates that the pressure head in the nodes is uniform and the results are acceptable, (Subhi and Auath, 1990)

Pressure Uniformity Coefficient (UC)

UC is a measurable index of degree of uniformity obtainable for any pipe sizes operating under a specified consideration. The data used to calculate the Uniformity Coefficient (UC) are the modeled pressure heads in the nodes.

$$UC = 100\left[1 - \frac{\sum_{i=1}^{nn}\left(\left| Pi - \frac{\sum_{i=1}^{nn} Pi}{n}\right| - 0.15\right)}{\sum_{i=1}^{nn} Pi}\right] \tag{10}$$

Where P_i: Pressure head at node i in (m) and nn: total number of nodes.
A uniformity coefficient of 85% or more is considered to be satisfactory (indicative of absolutely uniform application) (Michael 1978).

WORKING ALGORITHM OF THE COST FUNCTION

The following steps are used to calculate the cost of a network **Fig. (1).**

1- Number generated by GLOBE are read from the parameter file and converted to indices of pipe sizes that represent one network.
2- The network simulation model is started.
3- The actual cost of the network (Cost1) is calculated based on pipe cost.
4- From the output file of the simulation, the nodal pressure heads are extracted and the minimum pressure is identified to calculate the penalty cost (Cost2).
5- The total cost of the network (Cost1+Cost2) is passed to the response file.
6- If the total cost is optimal then stop. If not, the input file of the simulation is updated (only the diameters are changed) and repeat steps (2-6).

CASE STUDY: AL-KARADA WATER SUPPLY NETWORK

Al-Karada (R9) water supply network located at near the end zone of Al-Rasafa section on the eastern bank of the Tigris river was the case study. Water supply network R9 is supplied from the 9-Nissan treatment plant by the main transmission pipeline and from Al-kadisia treatment plant from Al-Karak section. R9 network supplies potable water to the sections (919,921,915,913,923 ,911,907,909,925) in Al-Karada district. This distribution system was laid by the SOBEA Company and in order to define these pipes from the distribution pipeline, it is called SOBEA pipelines, see **Fig.(2), Table (1) and Table (2)** shows the properties of this network .

RESULTS OF THE COMPUTER PROGRAM FOR OPTIMAL DESIGN

To satisfy the required pressure head of ($H_{req.}$=20 m) in this network, the effect of the proposed storage tank (R9) was considered. Constructing this reservoir with a constant water level of 30 m may give the optimum design of the network as shown in **Table (3)** and **Fig. (3).** One may consider the difference in the diameter of the pipes that affect the network performance to achieve better pressure distribution as shown in **Fig. (4).** The uniformity of the optimum design for the hydraulic model is as shown in **Table (4).**

By using **Table (5),** the cost of the optimal design is 561,169,310 ID while the actual cost for the SOBEA design was 856,617,170 ID. The optimal design is 34.5% less than the actual cost of SOBEA.

EFFECT OF HAZEN-WILLIAM COEFFICIENT ON THE TOTAL COST

Water supply networks are considered important projects designed for longtime investments. The periodically design for such projects is within 50 years. Among the major factor considered in the design is the friction losses used to calculate the energy losses which affects the Hazen-William coefficient. The design diameters in the network are affected by Hazen-William coefficient. By using assuming a constant value for the Hazen-William coefficient for the whole network and Equation (7) the effect of the Hazen-William coefficient on the total cost is shown in **Table (6)** and **Fig. (5).** For example, if low value is assumed for this coefficient, the losses in the network will increase, which require using greater diameters to avoid losses in energy, as a result the network cost will increase and vice versa .The value of this coefficient is changed with time due to corrosion or incrustation in the internal pipe surface.

If the value of Hazen-William coefficient is greater than 130, the total cost is 600,893,300 ID, i.e., the penalty cost is zero and there is no effect for this coefficient on the total cost, only the cost of pipe is to be considered.

CONCLUSIONS

The following conclusions can be deduced:

1- It is possible to optimize networks with any kind of hydraulic facilities as long as network simulator is capable of handling it. Since global optimization method, work with any objective (cost) functions, they can also be efficiently be used to optimize not only design but also operation, maintenance and other aspects of water distribution networks.
2- Al-Karada water supply network is not within the optimal operating scheme.
3- The actual cost for SOBEA design pipes is greater than the optimal design by 34.5%.
4- The optimal design proposed by this study for Al-Karada water supply network has a minimum design pressure head of 20 m and uniformity indices as UC=99.565, σ =3.6508, and Cv =0.1543, i.e., uniform pressure distribution in pipe network.
5- The effect of Hazen-William coefficient (C) on the total cost of the network decreases linearly with increasing C up to 130, above this value the penalty cost will have no effect on the total cost.

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BWSA: Baghdad Water Supply Administration
R9: Al-Karada water Supply network
SOBEA: pipeline Engineering Company

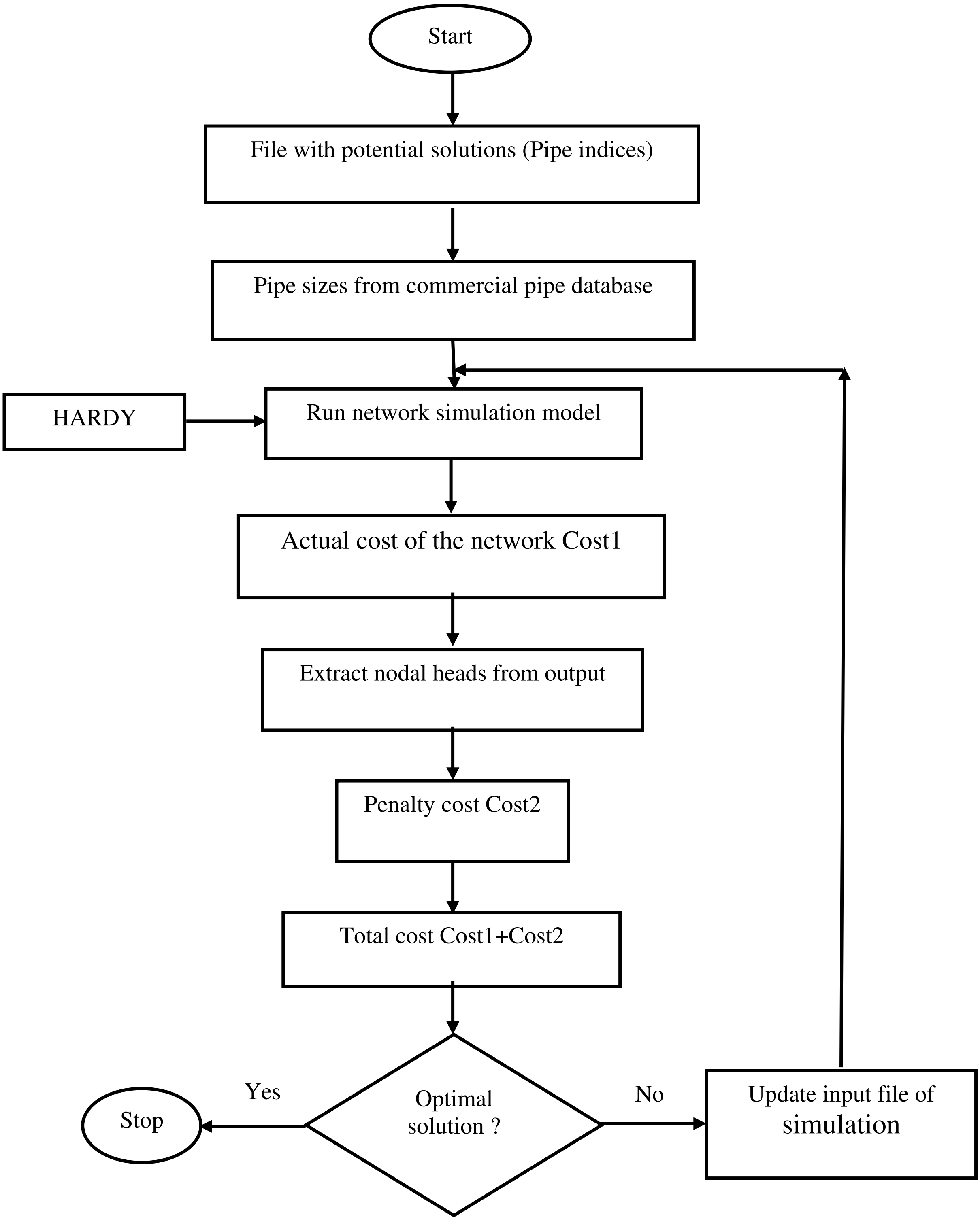

Fig. (1) Working Algorithm of the Cost Function

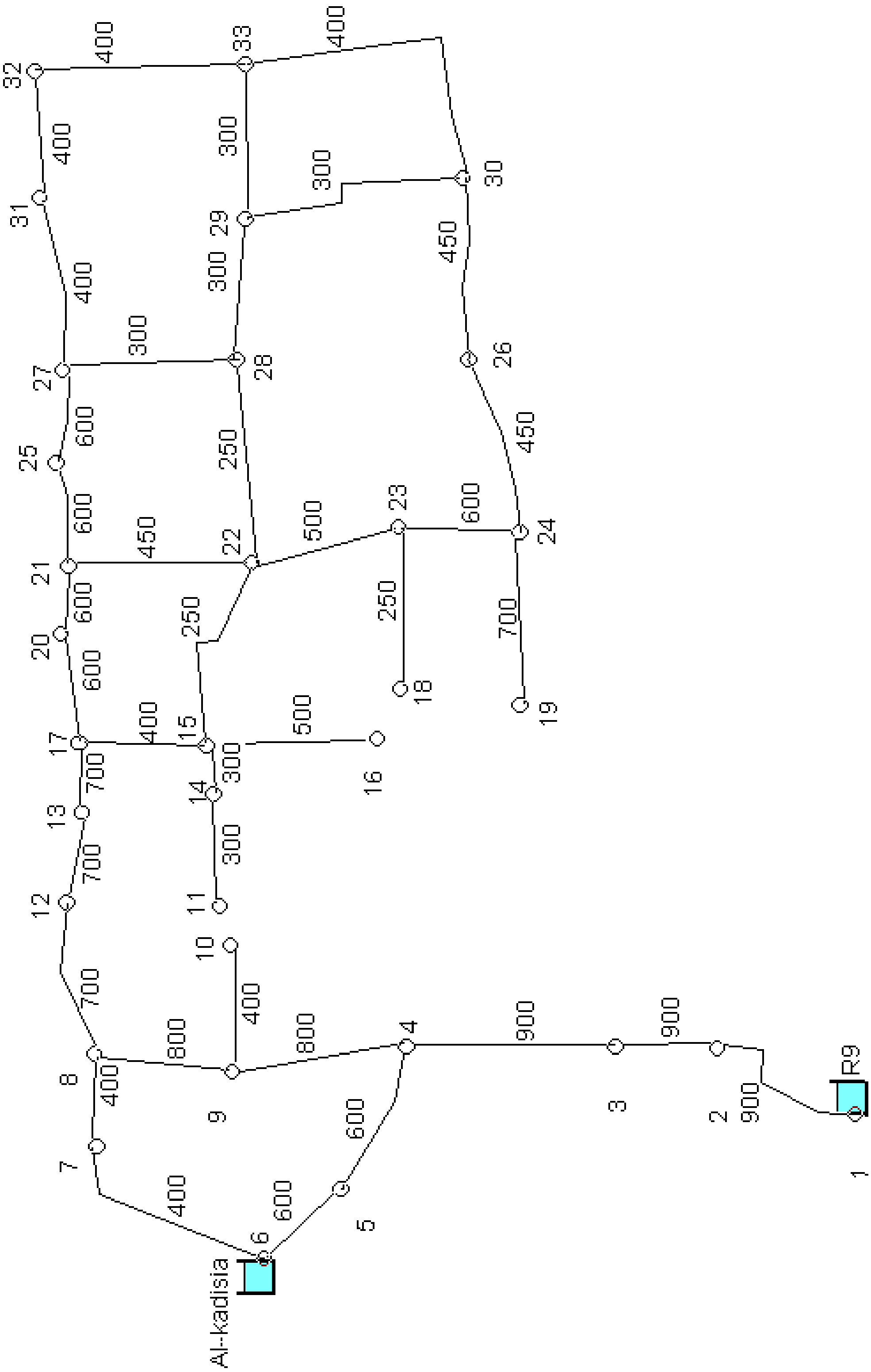

Fig.(2) Al-Karada Water Supply Network

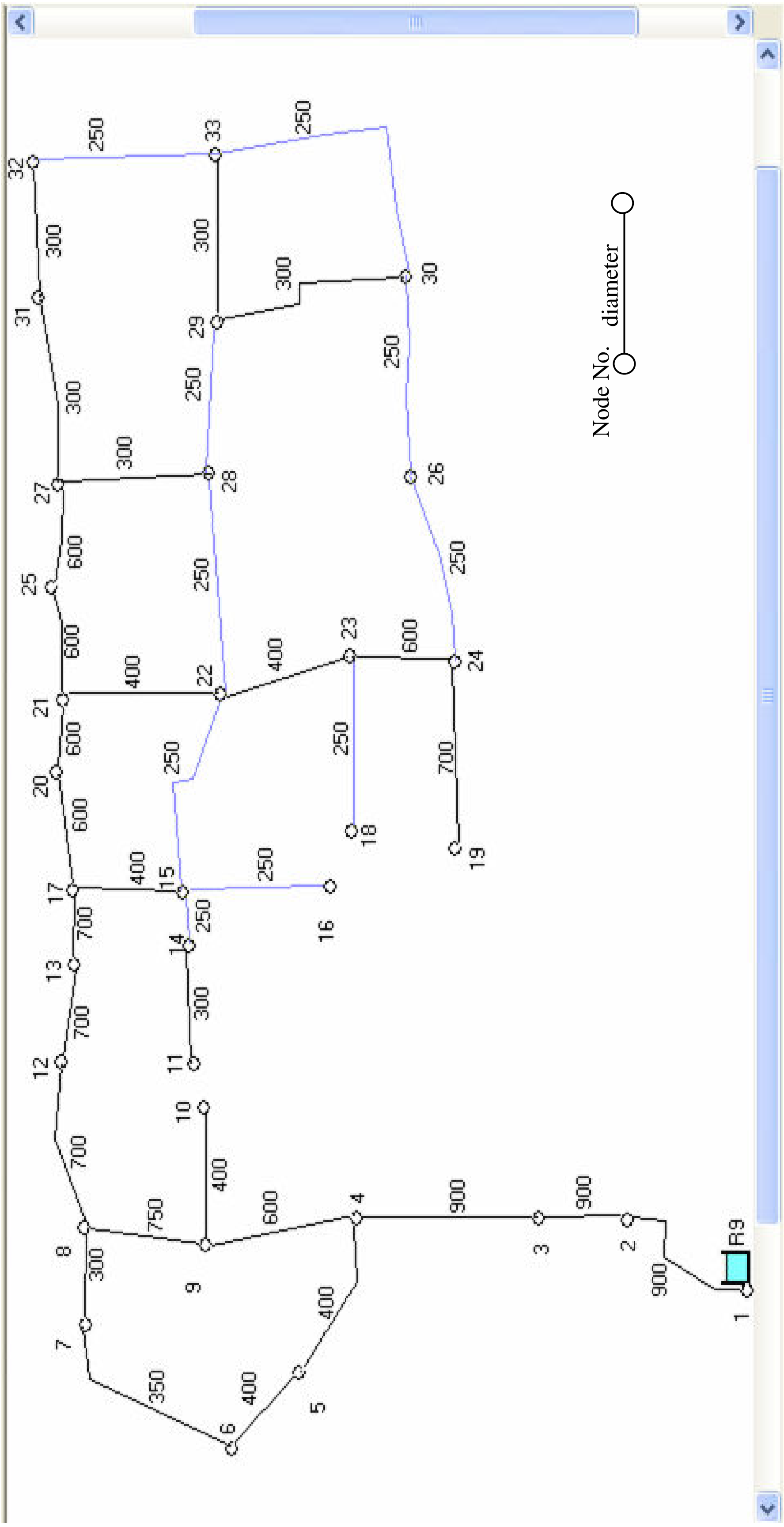

Fig. (3) Optimal design when R9 in operation

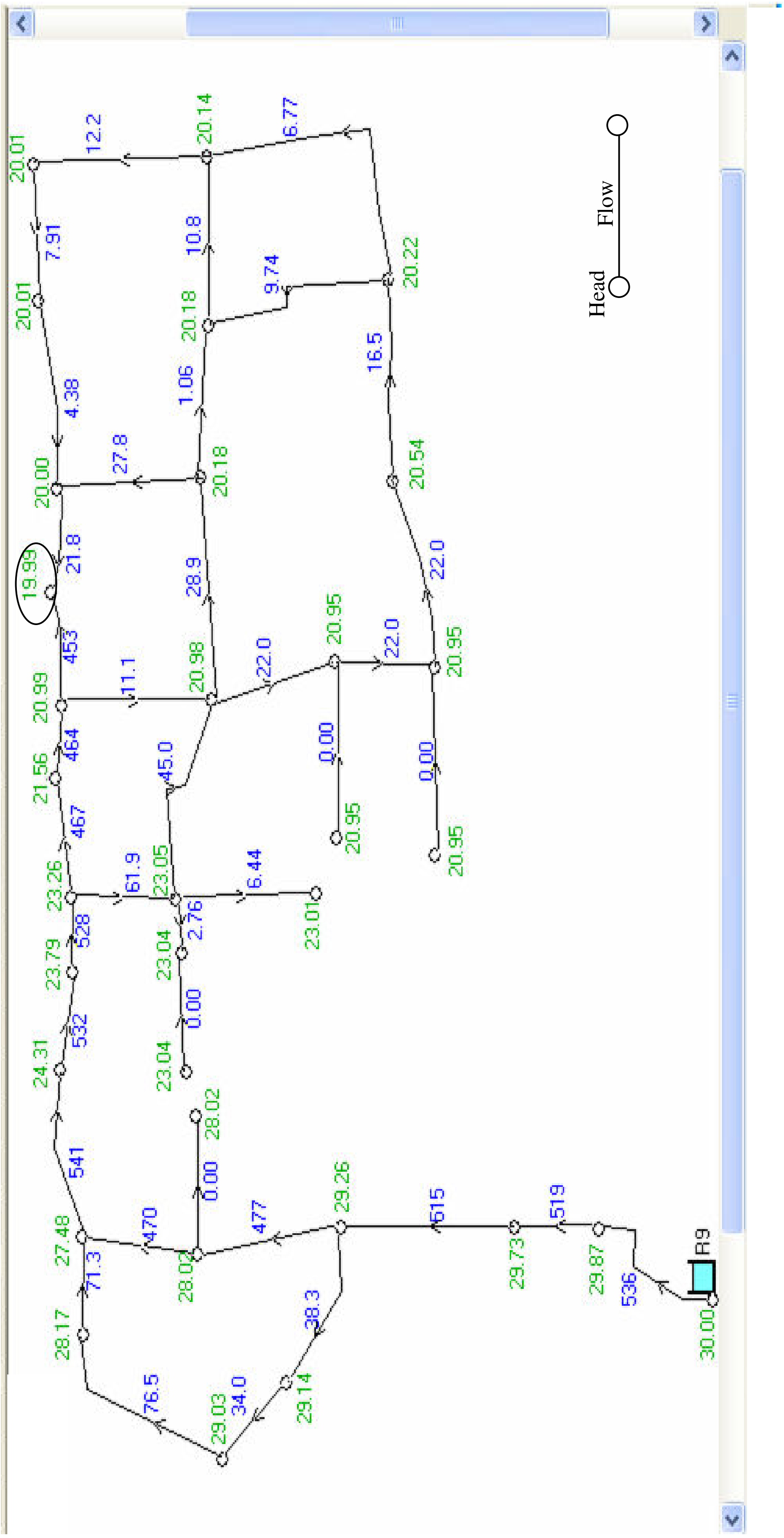

Fig. (4) Hydraulic analysis of the network (R9 in operation)

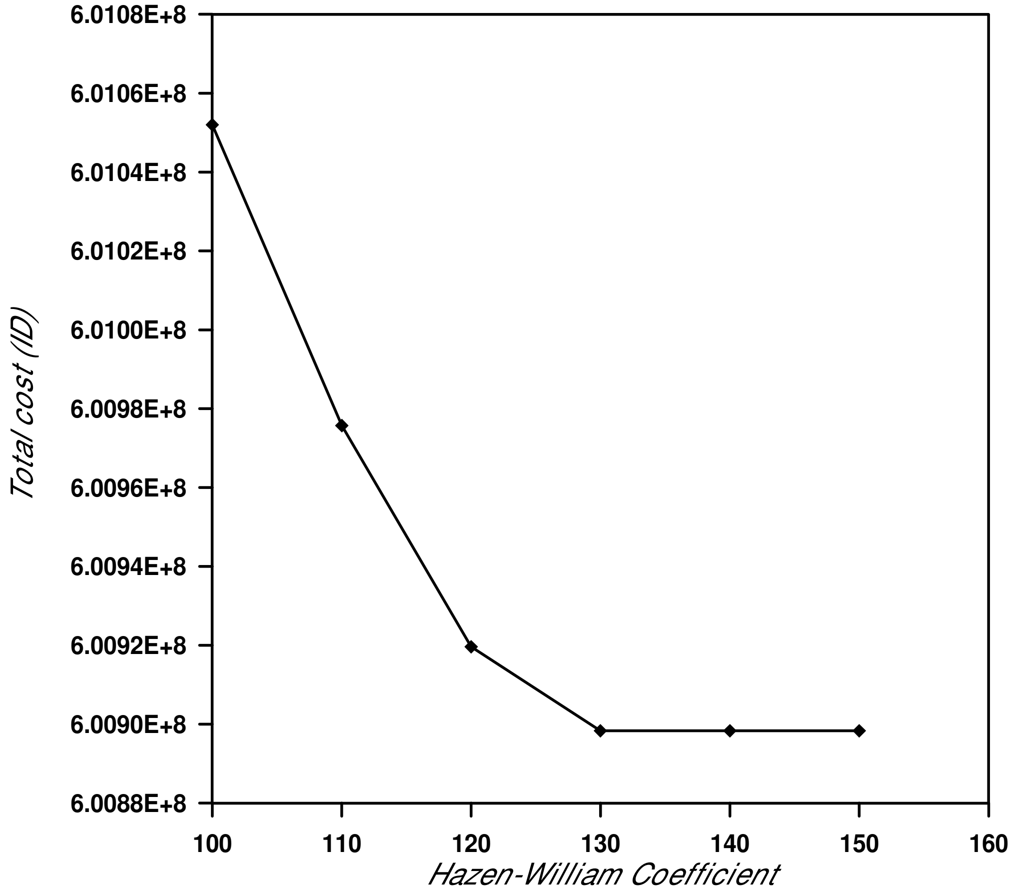

Fig. (5) Effect of the Hazen-William Coefficient on the Total Cost.

Table (1) Pipe Properties of Al-Karada Water Supply Network.

Pipe No.	From Node No.	To node No.	Pipe length (m)	Equivalent Length (m)	Pipe Diameter (mm)	Hazen-William coeff.
1.0	1.0	2.0	150.0	5.0	900	114.7
2.0	2.0	3.0	162.5	7.0	900	117
3.0	3.0	4.0	645.0	10.0	900	121.6
4.0	4.0	5.0	425.0	8.0	600	124.7
5.0	5.0	6.0	425.0	8.0	600	121.9
6.0	6.0	7.0	507.5	9.0	400	137.97
7.0	7.0	8.0	285.0	5.0	400	137.2
8.0	8.0	9.0	450.0	5.0	800	122.2
9.0	4.0	9.0	340.0	7.0	800	122.3
10.0	8.0	12.0	1212.5	13.0	700	124
11.0	9.0	10.	455.0	5.0	400	-
12.0	12.0	13.0	205.0	5.0	700	125.4
13.0	13.0	17.0	212.5	5.0	700	125.6
14.0	11.0	14.0	562.5	10.0	300	-
15.0	14.0	15.0	547.5	8.0	300	132.3
16.0	17.0	15.0	325.0	5.0	400	131.45
17.0	15.0	16.0	450.0	7.0	500	121.85
18.0	17.0	20.0	525.0	7.0	600	127.1
19.0	20.0	21.0	175.0	3.0	600	127.1
20.0	15.0	22.0	805.0	8.0	250	137.5
21.0	18.0	23.0	565.0	14.0	250	-
22.0	19.0	24.0	497.0	10.0	700	-
23.0	21.0	22.0	550.0	10.0	450	130.7
24.0	22.0	23.0	275.0	5.0	500	128.9
25.0	23.0	24.0	300.0	8.0	600	125.9
26.0	21.0	25.0	320.0	8.0	600	126.7
27.0	22.0	28.0	695.0	10.0	250	137.3
28.0	24.0	26.0	462.0	10.0	450	133.47
29.0	25.0	27.0	320.0	8.0	600	115.1
30.0	26.0	30.0	677.50	10.0	450	128.2
31.0	27.0	28.0	400.0	11.0	300	131.2
32.0	28.0	29.0	205.0	5.0	300	131.3
33.0	29.0	30.0	655.0	15.0	300	137.5

Table (1) Continue

Pipe No.	From Node No.	To node No.	Pipe length (m)	Equivalent Length (m)	Pipe Diameter (mm)	Hazen-William coeff.
34.0	27.0	31.0	630.0	10.0	400	126.9
35.0	29.0	33.0	585.0	15.0	300	128.8
36.0	30.0	33.0	980.0	40.0	400	128
37.0	31.0	32.0	75.0	5.0	400	121.7
38.0	32.0	33.0	547.5	12.0	400	140.3

Table (2) Nodes Properties of Al-Karada Water Supply

Node No.	Depth from Surface ground level (m)	No. of pipes connected to node	Draw-off from the node (m^3/s)	Supply to the node (m^3/s)
1.0	1.5	1.0	0	0.1608
2.0	1.4	2.0	0.03172	0
3.0	1.3	2.0	0.00985	0
4.0	1.4	3.0	0	0
5.0	1.5	2.0	0.008	0
6.0	1.3	2.0	0	0.08041
7.0	1.5	2.0	0.0112	0
8.0	1.4	3.0	0	0
9.0	1.4	3.0	0.01431	0
13.0	1.2	2.0	0.007587	0
14.0	1.0	2.0	0.006	0
15.0	1.5	4.0	0.0167	0
16.0	1.0	1.0	0.013976	0
17.0	1.0	3.0	0	0
18.0	1.1	1.0	0	0
19.0	1.0	1.0	0	0
20.0	1.2	2.0	0.007387	0
21.0	0.9	3.0	0	0
22.0	1.0	4.0	0.011314	0
23.00	1.0	3.0	0	0
24.0	1.2	3.0	0	0
25.0	1.0	2.0	0.011913	0
26.0	0.9	2.0	0.011913	0

Table (2) Continue

Node No.	Depth from Surface ground level (m)	No. of pipes connected to node	Draw-off from the node (m^3/s)	Supply to the node (m^3/s)
27.0	1.5	3.0	0.0225	0
28.0	1.3	3.0	0	0
29.0	1.5	3.0	0	0
30.0	1.7	3.0	0	0
31.0	1.5	2.0	0.0153	0
32.0	1.7	2.0	0.009317	0
33.0	1.5	3.0	0.011646	0

Table (3) Optimal design of the network

Pipe No.	From node	To node	Design diameter (mm)	Pipe length (m)
1.0	1.0	2.0	900	150.0
2.0	2.0	3.0	900	162.5
3.0	3.0	4.0	900	645.0
4.0	4.0	5.0	400	425.0
5.0	5.0	6.0	400	425.0
6.0	6.0	7.0	350	507.5
7.0	7.0	8.0	300	285.0
8.0	8.0	9.0	750	450.0
9.0	4.0	9.0	600	340.0
10.0	8.0	12.0	700	1212.5
11.0	9.0	10.0	*	*
12.0	12.0	13.0	700	205.0
13.0	13.0	17.0	700	212.5
14.0	11.0	14.0	*	*
15.0	14.0	15.0	250	547.5
16.0	17.0	15.0	400	325.0

Table (3) Continue

Pipe No.	From node	To node	Design diameter (mm)	Pipe length (m)
17.0	15.0	16.0	250	450.0
18.0	17.0	20.0	600	525.0
19.0	20.0	21.0	600	175.0
20.0	15.0	22.0	250	805.0
21.0	18.0	23.0	*	*
22.0	19.0	24.0	*	*
23.0	21.0	22.0	400	550.0
24.0	22.0	23.0	400	275.0
25.0	23.0	24.0	400	300.0
26.0	21.0	25.0	600	320.0
27.0	22.0	28.0	250	695.0
28.0	24.0	26.0	250	462.0
29.0	25.0	27.0	500	320.0
30.0	26.0	30.0	250	677.50
31.0	27.0	28.0	300	400.0
32.0	28.0	29.0	250	205.0
33.0	29.0	30.0	300	655.0
34.0	27.0	31.0	300	630.0
35.0	29.0	33.0	250	585.0
36.0	30.0	33.0	250	980.0
37.0	31.0	32.0	250	75.0
38.0	32.0	33.0	250	547.5

* Noted that these pipes can be neglected because there was no draw-off from the nodes which these pipes were connected to.

Table (4) uniformity Indicator results

Indicator	Result
Uniformity Coefficient (UC)	99.565
Standard deviation (σ)	3.6508
Coefficient of variance(Cv)	0.1543

Table (5) Prices of Commercial Diameters for Ductile Iron Pipes Including Rubber Joint for 1979 **

Diameter(mm)	Cost ID/m length	Diameter(mm)	Cost ID/m length
100	4,261	700	87,983
150	4,835	800	117,342
200	5,294	900	139,600
250	5,722	1000	166,750
300	18,220	1200	264,988
350	19,942	1300	268,550
400	25,197	1400	345,479
450	47,417	1500	356, 488
500	51,155	1600	390,000
600	66,007		

** This table was provided from the Baghdad Water Supply Administration (BWSA) which was very important for calculation of the actual cost of the optimal design.

Table (6) Effect of *Hazen-William* coefficient on the total cost

Hazen-William coefficient	Total cost (ID)
100	601,052,000
110	600,975,700
120	600,919,600
130	600,898,300
140	600,898,300
150	600,898,300