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HYDRAULIC CALIBRATION OF PIPE NETWORK MODEL USING AN IMPROVED GENETIC TECHNIQUE

معايرة هيدروليكية لشبكات الأنابيب بإستخدام طريقة جينية معدلة

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خلاصة

يهتم هذا البحث بعرض إمكانية برنامج حاسوب مستحدث يسمي SO-OPTIM تمت كتابته بلغة الفورتران بتطبيق مبادئ كل من الطريقة الخوارزمية الجينية المعدلة وحيدة الهدف وطريقة جديدة لحل شبكات الأنابيب في حالة السريان المستقر و ذلك في حل عدة مشاكل بشبكات توزيع المياه مع التركيز علي المعايرة الهيدروليكية لها و تشمل: تصميم شبكات المياه الجديدة ، إحلال بعض الأنابيب للشبكات العاملة لضبط الضغوط عليها، الجدولة المثلي لمضخات المياه، التشغيل الأمثل لمحطات المياه الموجودة علي الشبكات في حالة وجود الخزانات العالية عليها، التنظيم الأمثل للضغوط بالإضافة إلي المعايرة الهيدروليكية. تم التعبير عن كل مشكلة من المشاكل سابقة الذكر بدالة هادفة لا خطية في وجود مجموعة من القيود الهيدروليكية الخطية ولا خطية. طبقت عدة تعديلات علي الطريقة الخوارزمية الجينية البسيطة لزيادة سرعة الوصول إلي الحل الأمثل. أستخدم البرنامج في عمل معايرة لشبكتين: الأولي افتراضية ذات قياسات تركيبية حيث يقوم البرنامج بحساب قيم كل من معاملات هازن- ويليامز للإحتكاك خلال الأنابيب و قيم الإستهلاك عند العقد (نقط إتصال الأنابيب) ، بينما الشبكة الثانية هي شبكة مدينة دمنهور والتي تحتوي علي قياسات فعلية ويقوم البرنامج في هذه الحالة بإستنتاج قيم معاملات كل من هازن- ويليامز والفواقد الثانوية خلال الأنابيب.

ABSTRACT

This research paper exhibits the development and calibration capability of a computer optimization code, called SO-OPTIM. This code copes with several problems concerning water distribution networks which include: new design, rehabilitation, calibration; optimal scheduling of pumps, optimal operation of pump stations in the presence of water elevated tanks, and optimal pressure regulation. These problems are treated using a powerful genetic algorithm optimization technique which consists of a nonlinear objective functions subjected to both linear and non linear constraints. Different modifications were carried out in the simple genetic algorithm. The mentioned code links the optimization model and a hydraulic solver which applies the principles of a new technique used for pipe network analysis in steady state conditions. The proposed code is applied in two different water distribution networks. The first one is a hypothetical network with synthetic calibration data and the SO-OPTIM code is applied to perform extended period calibration. The second network is a real water distribution system of Damnhour city, Egypt, with actual data and field measurements and in this case the code performs steady state calibration.

1. INTRODUCTION

The successful application of mathematical models in analyzing and designing water distribution systems is highly dependent upon the accuracy of required input data. These data can be accurately determined except pipe roughness coefficients and nodal demands. Difficulties associated with cost of performing the measurements and reliability of the measured data often dictate estimation of these parameters through model calibration. Therefore, model calibration means the adjustment of the estimated parameters (i.e., pipe roughness coefficients and nodal demands) until model results closely approximate the observed conditions

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measured from field data (Lingireddy and Ormsbee 2002).

Calibration can be divided into steady state calibration and extended period (quasi-steady state) calibration. In the steady state one, the roughness coefficients, independent on the time, are adjusted to perform simulation between both the observed pressures and flows and the calculated ones. In the extended period calibration, the distribution of demands, dependent on the time, are adjusted to match time varying pressure and flows. Calibration of models can be grouped into three categories: iterative procedure models (trial and error), explicit models (hydraulic simulation), and implicit models (optimization), Walski et al. (2008). An implicit model based on an improved genetic algorithm technique is proposed through this study.

Anderson and Al-Jamal (1995) presented a methodology using Linear programming method for simplification of hydraulic networks and fitting parameters approach. A comparison was carried out between the two alternative approaches of simplification, element- by-element and the fitting parameters approaches. Savic and Walters (1995) used genetic algorithm to calibrate a real network model for the Danes Castle Zone of Exeter city, U.K. A comparison was performed between results of GA calibration tool and the trial-and-error method. Solomatine (1998, 1999) presented two examples of global optimization (GO) practical use; the first one in hydrological model calibration and the latter in pipe network optimal design. A comparison was performed between various GO algorithms in their performance and most efficient algorithms. Greco and Giudice (1999) used a nonlinear optimization algorithm in combination with a standard off-the-shelf network solver for calibrating hydraulic network models. Pipe roughnesses were adjusted until agreement occurred between observed and predicted pressures assuming all nodal demands were known. Lingireddy and Ormsbee (2002) presented a genetic algorithm optimization approach to calibrate a network model for pipe roughness coefficients, spatial and temporal adjustment factors. A parametric study was conducted to study the influence of population size, probabilities of crossover and mutation on the performance of the genetic algorithm. The mathematical code was applied on two water distribution network models with synthetic calibration data. Wu et al. (2002) compared between the two types of calibration, the traditional way (trial and error) and the calibration using genetic algorithm. Three different types of calibration objective functions were exhibited. Bartolin and Martinez (2003) presented ArcView® GIS extension called GISRed which was used in water network modeling. Capabilities of GISRed were mentioned which contain simulating, analyzing, and retrieving the actual network status under certain conditions as well as calibration of pipe network using genetic algorithm. Wu and Walski (2005) developed a genetic algorithm optimization tool for calibrating water distribution models. An approach for effectively applying the optimized calibration methods was proposed. Vassiljev et al. (2007) used pressure differentials to calibrate the operational water distribution system containing thousands of pipes with different ages. The proposed approach eliminates significantly the influence of leakages as well. Colombo and Giustolisi (2007) introduced and explained a data-driven strategy for conducting water distribution network pipe roughness calibration. The proposed approach is based on solving the inverse calibration problem and then applying Evolutionary Polynomial Regression (EPR) which depends on genetic algorithm in order to encapsulate network behavior. Kumar et al. (2009) presented a formal procedure based on K-means clustering algorithm for grouping the pipes having similar roughness characteristics. Graph-theoretic concepts were used to reduce the dimensionality of the problem. Behzadian et al. (2009) used both multiobjective genetic algorithm and adaptive neural networks to determine optimal sampling locations for installing pressure loggers in pipe networks, taking into consideration parameters uncertainty. The objectives of the study were to maximize the calibrated model accuracy and minimize the number of sampling devices.

The main purpose of this research paper is to illustrate only one capability of SO-OPTIM code in calibration of two pipe networks. The first one is a hypothetical network with synthetic calibration data and the second network is the real network of Damnhour city, Egypt.

2. SOLUTION METHODOLOGY

A FORTRAN code named SO-OPTIM has been written for applying the principles of both the genetic algorithm and a new technique for performing pipe network hydraulic analysis in steady state conditions. This code treats several problems concerning water distribution networks which include: new design, rehabilitation, calibration; optimal scheduling of pumps, optimal operation of pump stations in the presence of water elevated tanks, and optimal pressure regulation.

2.1. Pipe Network Hydraulic Analysis for Steady State Conditions

Both Linear Theory Method (LTM) and Extended Linear Graph Theory (ELGT) are linked to get a new technique which could be used for the analysis of pipe networks. This technique differs from other linear theory methods in the system formation of linear equations and solution procedures. The solution algorithm used in this technique is independent on initial pipe flows estimation, where a power law equation is used to update the pipe flows in successive iterations. The proposed method has been extended to deal with complex systems including control devices such as pumps, pressure reducing valves (PRVs), pressure sustaining valves (PSVs), and check valves (CVs). Formulation of this new technique is programmed in a FORTRAN subroutine named SFLOW as given by El-Ghandour (2009).

2.2. Steps of the Genetic Algorithm Solution through SO-OPTIM Code

A brief description of the genetic algorithm solution through SO-OPTIM code can be described in the following steps:

1. *Generation of initial population*: this step generates an initial population of chromosomes randomly, ranges between 50 and 200, put them in the father pool. Every chromosome within the created population consists of a number of genes equal to the number of unknown variables.
2. *Hydraulic analysis of each network*: for every chromosome located within the father pool, the hydraulic analysis is applied using SFLOW subroutine to calculate the nodal pressure heads and the flow rates through pipes. The nodal pressure heads and the pipe flow velocities for every chromosome are compared with the minimum allowable pressure heads and the maximum allowable velocity, and any pressure head or velocity deficits are noted.
3. *Computation of objective function*: for each chromosome in the population, this step computes the objective function which describes the problem under consideration.
4. *Computation of penalty*: the GA assigns a penalty for each chromosome if a suggested solution does not satisfy one or more of the constraints. In this study, the developed adaptive penalty function given by Yassen (2007) is applied.
5. *Computation of total objective function*: in this step, the total objective function is computed which is the sum of the objective function computed in step (3) plus the penalty value computed in step (4).

6. *Computation of fitness*: all problems which handled by SO-OPTIM code are treated as an objective function minimization, then the fitness for each chromosome is taken as the inverse of the total objective function (step 5). The probability of every chromosome P_i is calculated according to the following equation:

$$P_i = \frac{F_i}{\sum_{i=1}^{NSO} F_i} \qquad (1)$$

in which, F_i = fitness of chromosome i, and NSO = number of chromosomes in the father pool.

7. *Generation of a new population*: this procedure uses the selection, crossover, and mutation operators as follows:
   - Selection: selects two chromosomes randomly according to its probability, Eq. (1), using the roulette wheel method (Goldberg 1989).
   - Crossover: generates uniform random number between 0 and 1 and compares this number with crossover ratio. If it is smaller than the crossover ratio, the crossover on the two chromosomes is applied to create one child chromosome using uniform crossover between different genes, otherwise the fittest chromosome is taken and put in the children pool.
   - Mutation: for each gene within the children chromosome iteratively generates a uniform random number between 0 and 1, if it is smaller than the mutation ratio, mutation has to be applied on this gene. This mutation operates randomly by a new value for this gene.

   The previous steps have to be repeated iteratively for each new created chromosome in the children pool.

8. Production of successive generation: steps from 2 to 7 are repeated to generate successive generations.

2.3. Modifications in the Simple Genetic Algorithm Technique

Several modifications were tacked in the simple genetic algorithm to improve the performance of this method as a tool for greatly increasing the algorithm convergence. These modifications are as follows:

- A modified adaptive penalty function given by Yassen (2007) is adopted through this study;
- Two types of replacement strategy have to be considered. The first one is to replace the weakest string in the children pool with the fittest one from the father pool. The second type is to select two strings randomly from old generation and the fittest one (smaller total cost plus total penalty) is determined. This string replaces the first string in the children pool if it is the fittest otherwise the first children pool string is remained in its location. This procedure is repeated for all strings in the children pool;
- A real value for coding the decision variables is used instead of using binary or gray coding;
- A roulette wheel method is considered for selection of the chromosomes from the father pool;
- Three types of crossover are adopted: one point crossover, two points crossover, and uniform crossover; and
- A modified uniform mutation is used.

2.4. Code Structure

SO-OPTIM code consists of the main program and nine subroutines. A brief description of this code is given as follows:

- *Main program*: the main program receives input data file. The main program also performs certain jobs such as applying iteratively the GA operators (selection, crossover, and mutation) to produce the new generations. The new second generation has to be settled in the father pool instead of the old generation, and then the GA operators have to be applied again on that new generation. This procedure will be repeated to a pre-specified number of maximum generations. Finally, the main program presents the results of the optimal solution. For this optimum solution, also the main program can present values of nodal heads, and flow through each pipe.
- *Subroutine (COMPLETE)*: called iteratively in the main program to calculate the objective function, fitness, probability for each chromosome, and the summation of all probabilities. These calculations have to be repeated for every new generation.
- *Subroutine (IGREAT)*: called iteratively in the main program to choose a chromosome from the father pool using a roulette wheel selection. Every chromosome has a chance for selection proportional to its corresponding probability/fitness.
- *Subroutines (DESIGN, CALIB, PUMPSC, PUMPST, and PRESSREG)*: each subroutine from these five subroutines corresponds to each function of SO-OPTIM code which contains new design or rehabilitation, calibration, optimal scheduling of pumps, optimal operation of pump stations, and optimal pressure regulation. Each subroutine is called from the main program to generate the initial population of (*NSO*) chromosomes randomly and puts them in the father pool and calculates the total objective function, which equals the values of objective function plus all penalties.
- *Function (RAN3)*: called iteratively in the DESIGN, CALIB, PUMPSC, PUMPST, and PRESSREG subroutines to generate random number between 0 and 1. Any input integer is named the seed number.
- *Subroutine (SFLOW)*: called iteratively for every new chromosome in the DESIGN, CALIB, PUMPSC, PUMPST, and PRESSREG subroutines to determine the nodal heads and the discharge through pipes using the new technique.

Figure (1) shows the general flow chart for the SO-OPTIM code.

3. MATHEMATICAL FORMULATION FOR CALIBRATION

This section presents an optimization approach to calibrate any network model for pipe roughness coefficients; pipe secondary losses coefficients; and spatial and temporal demand adjustment factors. Implicit model (optimization model) was adopted in this study. Then, the calibration problem has to be formulated as a nonlinear optimization problem consisting of a nonlinear objective function subjected to both linear and nonlinear equality and inequality constraints.

3.1. Objective Function

The used objective function, in case of extended period calibration, will be formulated to minimize the square of the differences between observed and predicted values of pressures and flows. Mathematically, this may be expressed as follows (Lingireddy and Ormsbee 2002):

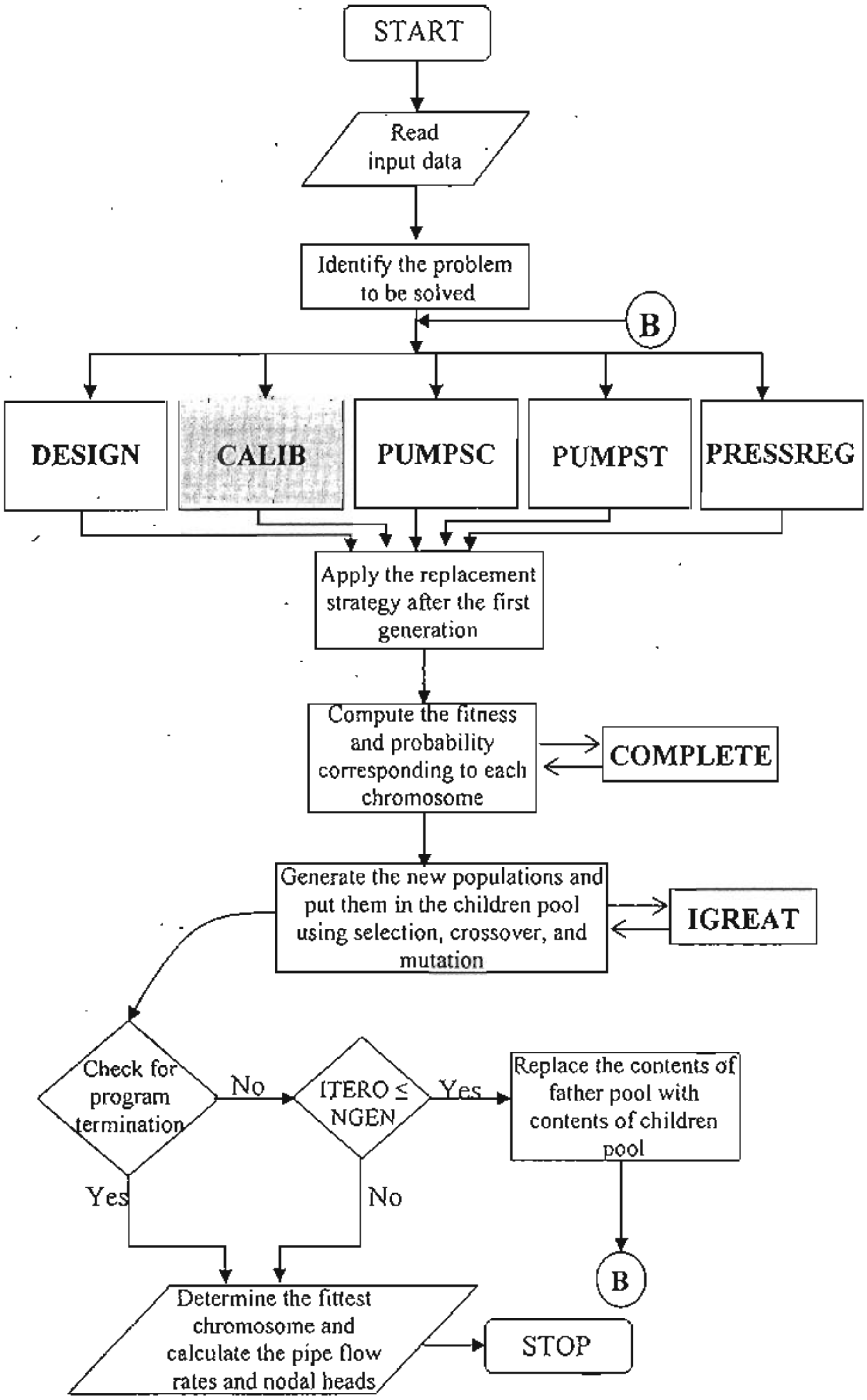

Figure (1): General flow chart for the SO-OPTIM code

$$Obj = \sum_{t=1}^{T}\left[\varphi\sum_{i=1}^{I}\left(OP_{it} - PP_{it}\right)^2 + \zeta\sum_{j=1}^{J}\left(OQ_{jt} - PQ_{jt}\right)^2\right] \quad (2)$$

in which, OP_{it} and PP_{it} = measured and simulated pressures at junction node i at time t; OQ_{jt} and PQ_{jt} = measured and simulated flows in pipe j at time t; I = number of junction nodes with available pressure readings; J = number of pipes with available flow measurements; T = number of time increments during an extended period simulation (equal zero for steady state simulation); and φ and ζ= normalization weights.

The following expressions are considered for normalization weights (Lingireddy and Ormsbee 2002):

$$\varphi = \{100 / Max(OP_{it})\}^2 \quad for\ i = 1,...,I;\ t = 1,...,T \tag{3}$$

$$\zeta = \{100 / Max(OQ_{jt})\}^2 \quad for\ j = 1,...,J;\ t = 1,...,T \tag{4}$$

The reason of using both φ and ζ in Eq. (2) is the difference in units between the pressure and flow, therefore these scaling factors (φ and ζ) can be used to make an agreement between both terms in Eq. (2) and increase the efficiency of optimization algorithms, or in other words a transformation of the multiple objectives into single objective problem.

3.2. Constraints

Constraints can be classified into three groups, implicit bound constraints explicit bound constraints and implicit system constraints.

The implicit bound constraints (Mays 2000) may include both nodal pressure heads and pipe flows bound constraints. These constraints can be used to ensure that the resulting calibration does not produce unrealistic pressures or flows as a result of the model calibration process. Mathematically, these constraints can be expressed as follows:

$$H_{j\,min} \leq h_j \leq H_{j\,max}, \quad j = 1, \ldots\ldots n \tag{5}$$

$$q_{i\,min} \leq q_i \leq q_{i\,max}, \quad i = 1, \ldots\ldots p \tag{6}$$

in which, h_j = pressure head at node j; $H_{j\,min}$ and $H_{j\,max}$ = minimum and maximum allowable heads at node j; q_i = flow rate in pipe i; $q_{i\,min}$ and $q_{i\,max}$ = minimum and maximum allowable flow rates at pipe i, and n and p = number of nodes and pipes of the network respectively.

The explicit bound constraints (Mays 2000) can be used to set limits on explicit decision variables of the calibration problems. Normally, these variables will include the roughness as well as secondary losses coefficients of each pipe and the demand at each node. Mathematically, these constraints can be expressed as follows:

$$(C_{HW})_{iL} \leq (C_{HW})_i \leq (C_{HW})_{iU}, \quad i = 1, \ldots, p \tag{7}$$

$$k_{siL} \leq k_{si} \leq k_{siU}, \quad i = 1, \ldots, p \tag{8}$$

$$Q_{jL} \leq Q_j \leq Q_{jU}, \quad j = 1, \ldots, n \tag{9}$$

in which, $(C_{HW})_i$ = Hazen – Williams coefficient for pipe i; $(C_{HW})_{iL}$ and $(C_{HW})_{iU}$ = lower and upper allowable Hazen – Williams coefficients for pipe i; k_{si} = secondary losses coefficient for pipe i, k_{siL} and k_{siU} = lower and upper allowable secondary losses coefficient for pipe i; p = number of pipes through the network under study; Q_j = demand/flow at node j; Q_{jL} and Q_{jU} = lower and upper allowable nodal demands/flows at node j; and n = number of nodes through the network under study.

The implicit system constraints (Yassen 2007) include nodal conservation of mass and conservation of energy. Mathematically, these constraints can be expressed as follows:

1. Nodal conservation of mass: inflow and outflow must be balanced at each junction node as follows:

$$\sum Q_{in} - \sum Q_{out} = Q_e \tag{10}$$

for each junction node (other than the source, i.e. excluding reservoir and tanks)

in which, Q_{in} = flow into the junction, Q_{out} = flow out of the junction, Q_e = external inflow or demand at the junction node.

2. Conservation of energy: head loss around a closed loop must equal zero or pump energy head if there is a pump.

$$\sum h_f = zero \qquad (11)$$

(around each loop in case of there is no pump)

$$\sum h_f = E_p \qquad \text{(if there is a pump)} \qquad (12)$$

in which, h_f = head loss due to friction in a pipe, and E_p = the energy supplied by a pump.

3.3. Decision Variables

The decision variables include pipe roughness coefficient of each pipe and demand at each node. Instead of using the values of nodal demands as decision variables, multiplier associated for each average nodal demand can be used. Value for each multiplier ranges from 0.8 to 1.2 (Wu et al. 2002). The average nodal demands are multiplied by the computed values of multiplier to give the actual nodal demands. Multiplier corresponds to each nodal demand at certain interval of time is called spatial multiplier. This multiplier changes from one interval of time to another. When multiplier depends on time, it is called temporal multiplier. Inclusion of an unknown roughness coefficient for each pipe line and an unknown demand multiplier for each junction node gives an excessive number of decision variables, for even a small water distribution system. Then, to reduce the number of decision variables, pipelines having the same physical and hydraulic characteristics can be grouped in one calibration link, with one roughness coefficient for all pipes in the same group. Also, all junction nodes that have the same demand pattern and within the same topological area can be aggregated as a one calibration junction with the same multiplier. In this way, the number of decision variables associated with the roughness coefficients can be reduced to the total number of groups of pipes in the distribution system. Also, the number of decision variables associated with the nodal multipliers can be reduced to the total number of groups of nodes having the same demand pattern.

Minimization of the objective function, Eq (2), subjected to constraints mentioned before can be performed using SO-OPTIM code including CALIB subroutine. Figure (2) shows the flow chart for the calibration subroutine. This flow chart corresponds to the extended period calibration. In case of steady state calibration, $N_{interval}$ in Figure (2) is equal to one whereas in the extended period calibration it is equal to the number of time intervals throughout the day, for every hour.

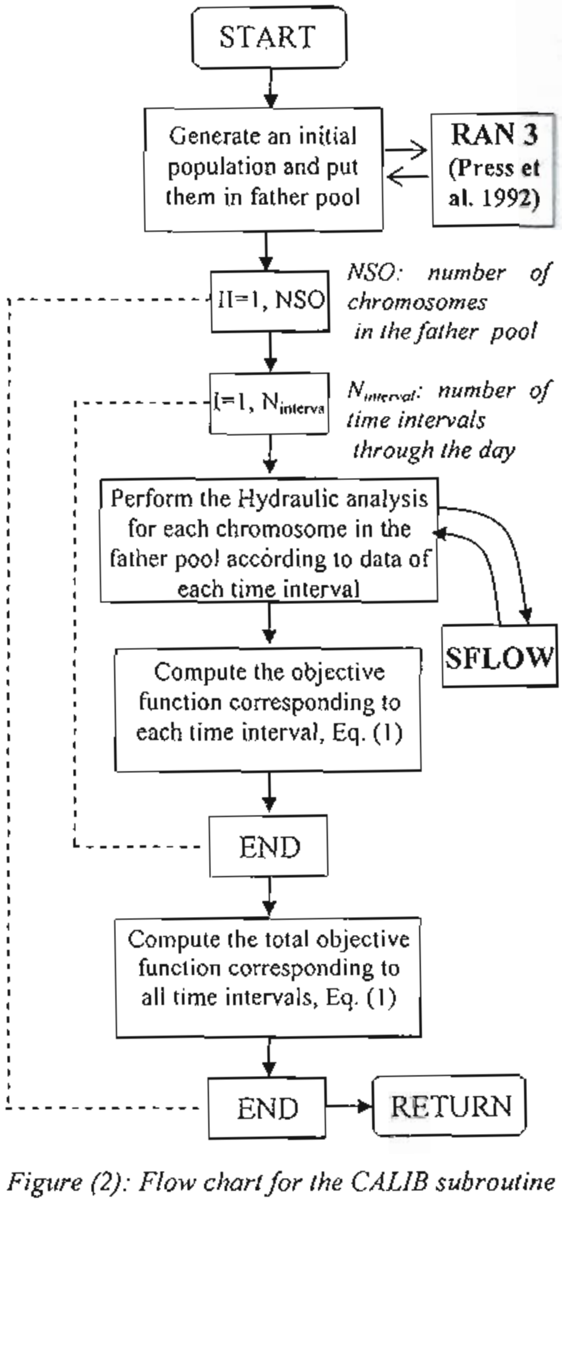

Figure (2): Flow chart for the CALIB subroutine

4. APPLICATIONS

Two cases have been considered in this study, hypothetical water distribution network and a real network of Damnhour city, Egypt.

4.1. Hypothetical Water Distribution Network

The usage of this case is to verify SO-OPTIM code against simple pipe network. The test case is a hypothetical water distribution network. The network consists of 14 pipes, 11 nodes, one well at node (1) with water level equals 1000 ft, one elevated storage tank at node (11) with water level equals 1150 ft, and one pump at pipe number (1). The pump head-discharge curve has an equation $h_p = -2.51\ q^2 + 16.71\ q + 155$, where, q denotes flow through pump and h_p is the corresponding dynamic head. The layout of the network is shown in Figure (3) (Watters 1984). All data about the network are presented in Table (1) including pipe data and nodal requirements.

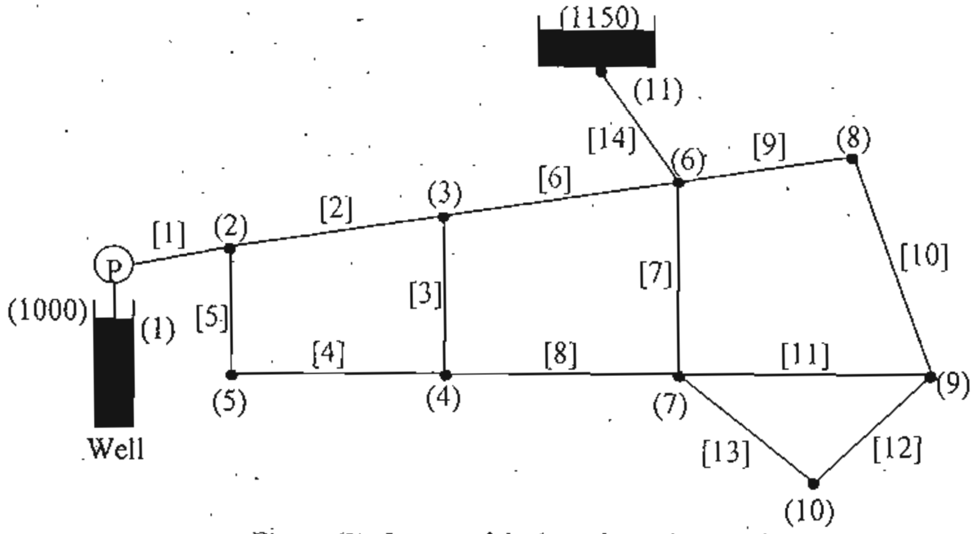

Figure (3): Layout of the hypothetical network (After Watters 1984)

Table (1): Pipe data and nodal requirements for a hypothetical network

Pipe no.	Diameter (in)	Length (ft)	Hazen – William coefficient	Node no.	Demand (cfs)	Level from specified datum (ft)
1	12.0	1000.0	110.0	1	----	1000.0
2	8.0	1000.0	90.0	2	0.0	1050.0
3	10.0	1200.0	120.0	3	0.5	1050.0
4	10.0	900.0	110.0	4	0.5	1050.0
5	10.0	1000.0	90.0	5	-1.5	1050.0
6	8.0	1000.0	100.0	6	0.0	1050.0
7	10.0	1400.0	100.0	7	1.0	1050.0
8	12.0	900.0	100.0	8	0.5	1050.0
9	8.0	1300.0	120.0	9	2.0	1050.0
10	10.0	1600.0	100.0	10	2.0	1050.0
11	8.0	1200.0	90.0	11	----	1050.0
12	10.0	1000.0	110.0			
13	10.0	1300.0	110.0			
14	12.0	500.0	90.0			

Table (1) gives the assumed Hazen – Williams roughness coefficient for each pipe. The network has been divided into four groups, each group has constant value of this coefficient. The value of roughness coefficient varies between 90 and 120 for the network. All demand nodes in the network can be assumed to have the same spatial pattern therefore; there is only one group for all demand nodes. In this hypothetical network, the calibration exercise is performed by making synthetic field data measurements using SFLOW code. These synthetic field data measurements contain two nodal pressure readings (nodes number 3 and 5) and one pipe flow rate (pipe number 3) corresponding to both an assumed set of temporal demand adjustment factors and estimated pipe roughness coefficients. Table (2) displays the assumed temporal demand adjustment factors and the corresponding synthetic field data. After noting the simulated data, both the temporal demand adjustment factors and pipe roughness coefficients are removed from the model. Consequently, the required aim is to predict both the temporal demand adjustment factors and pipe roughness coefficients with the aid of the simulated data shown in Table (2) using SO-OPTIM code. To calibrate the proposed model, specification of upper and lower bounds on both the group roughness coefficients and demand nodes group should be determined. These bounds are considered for group roughness coefficients as follows: (105 – 75), (115 – 85), (125 – 95), and (130 – 110) for groups 1, 2, 3, and 4 respectively whereas, (1.4 – 0.6) are considered as bounds for the demand nodes group.

The following parameters are considered through the SO-OPTIM code: population size = 50; maximum number of generations = 1000; crossover ratio = 0.7; mutation ratio = 0.1; the Idum number is taken (-1220); and uniform crossover is used.

Figure (4) shows the relationship between the calibration objective function corresponding to each generation and number of generations. From this figure it can be seen that, despite starting with random set of initial chromosomes, the SO-OPTIM code has obtained a fairly good solution in less than 150 generations. Table (3) displays the assumed and computed temporal demand adjustment factors and pipe roughness groups. It is evident from this table that the assumed and calculated values are nearly the same.

Table (2): Assumed boundary conditions for simulation

Time (hrs)	*Demand factor*	*Node number*	*Reading (ft)*	*Pipe number*	*Flow rate (cfs)*
0:00	*1.0*	*3* *5*	*99.44* *111.13*	*3*	*0.4863*
4:00	*1.2*	*3* *5*	*96.21* *109.92*	*3*	*0.6773*
8:00	*0.9*	*3* *5*	*100.88* *111.60*	*3*	*0.4077*
12:00	*1.4*	*3* *5*	*92.81* *108.57*	*3*	*0.9139*
16:00	*0.8*	*3* *5*	*102.15* *111.93*	*3*	*0.3425*
20:00	*0.6*	*3* *5*	*103.87* *111.98*	*3*	*0.2684*

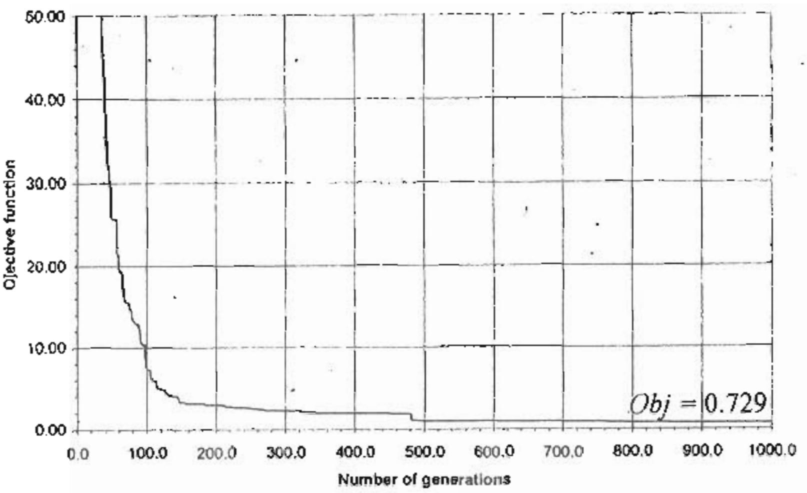

Figure (4): Relationship between objective function, Eq. (2) and number of generations for a hypothetical network

Table (3): Assumed and computed values for both demand factors and pipe roughness groups

Time	*Demand factors*		*Pipe roughness group number*	*Assumed (C_{HW})*	*Computed (C_{HW})*
	Assumed	*Computed*			
0.00	1.0	0.997	1	90	91.8
4.00	1.2	1.199	2	100	99.5
8.00	0.9	0.898	3	110	113.4
12.00	1.4	1.399	4	120	112.6
16.00	0.8	0.799			
20.00	0.6	0.608			

Table (4) exhibits the measured data used for the calibration and the corresponding simulated values. This table demonstrates the relative absolute average errors in both pressures and flows between the synthetic data and simulated ones as given by SO-OPTIM code, at assumed measurement locations according to the following equation:

$$error = \frac{1}{T}\sum_{t=1}^{T}\left\{\frac{\sum_{i=1}^{I}\frac{|OP_{it} - PP_{it}|}{OP_{it}} + \sum_{j=1}^{J}\frac{|OQ_{jt} - PQ_{jt}|}{OQ_{jt}}}{I + J}\right\}$$

in which, OP_{it} and PP_{it} = measured and simulated pressures at junction node i at time t; OQ_{jt} and PQ_{jt} = measured and simulated flows in pipe j at time t; I = number of junction nodes with available pressure readings; J = number of pipe with available flow measurements; and T = number of time increments during an extended period simulation (equal zero for steady state simulation).

The relative absolute average error is found equal to 0.22 %. This small value of error displays the efficiency of the SO-OPTIM code.

Table (4): Values of synthetic data and corresponding simulated results at assumed measurement locations

Time	Node No.	Pressure head readings		Pipe No.	Flow rate	
		measured (ft)	Simulated (ft)		measured (cfs)	Simulated (cfs)
0.00	3 5	99.44 111.13	99.71 110.88	3	0.4863	0.4861
4.00	3 5	96.21 109.92	96.53 109.60	3	0.6773	0.6783
8.00	3 5	100.88 111.60	101.11 111.37	3	0.4077	0.4078
12.00	3 5	92.81 108.57	93.24 108.21	3	0.9139	0.9141
16.00	3 5	102.15 111.93	102.32 111.72	3	0.3425	0.3439
20.00	3 5	103.87 111.98	103.97 111.84	3	0.2684	0.2694

4.2. Real Water Distribution Network of Damnhour City

Damnhour city is the capital of El-Beheira governorate where it mediates the governorate. Damnhour city lies between Rasheed branch of the River Nile at the west and Cairo - Alexandria agricultural road at the east. Damnhour city is at 64 kilometers distance away from city of Alexandria and at 160 kilometers distance from Cairo on the agricultural road. The city is bordered from the east with El-Khandak canal, the agricultural road of Cairo – Alexandria from the west, and there are agricultural lands on north and south borders.

Figure (5) shows Damnhour city network which contains 256 pipes and 193 nodes. All data and field measurements are listed in Damnhour master plan (2006). Locations of measurement points are only at the eight pipe lines that connect the Damnhour city network with the rural network. In the present mathematical model, these eight locations are a separation positions between the city network and the rural network, which they are shown in Figure (5) by big solid circles. The available field measurements at these locations are flow rates and pressure heads. In this model, the flow measurements at the eight separation points are considered as fixed grade points (boundary conditions) of known outflows from the city network to the rural network. Measurements of flows and pressure heads of these eight nodes are listed in Table (5). The pressure measurement values are considered as a reference for comparing the results of calibration. Therefore, the second term in the objective function, Eq. (2), which represents the square differences between predicted and observed flows, is removed. The adjustable model parameters are pipe roughness coefficients and secondary loss coefficients. These values are independent on the time, and then the extended calibration is transferred to steady state calibration.

Based on the discussion before, the objective function, Eq. (2) can take the following form:

$$Obj = \sum_{i=1}^{I}\left(OP_{it} - PP_{it}\right)^2 \qquad (14)$$

Calibration aims to adjust the values of both roughness and secondary losses coefficients by minimization of the objective function, Eq. (14), under the implicit system constraints.

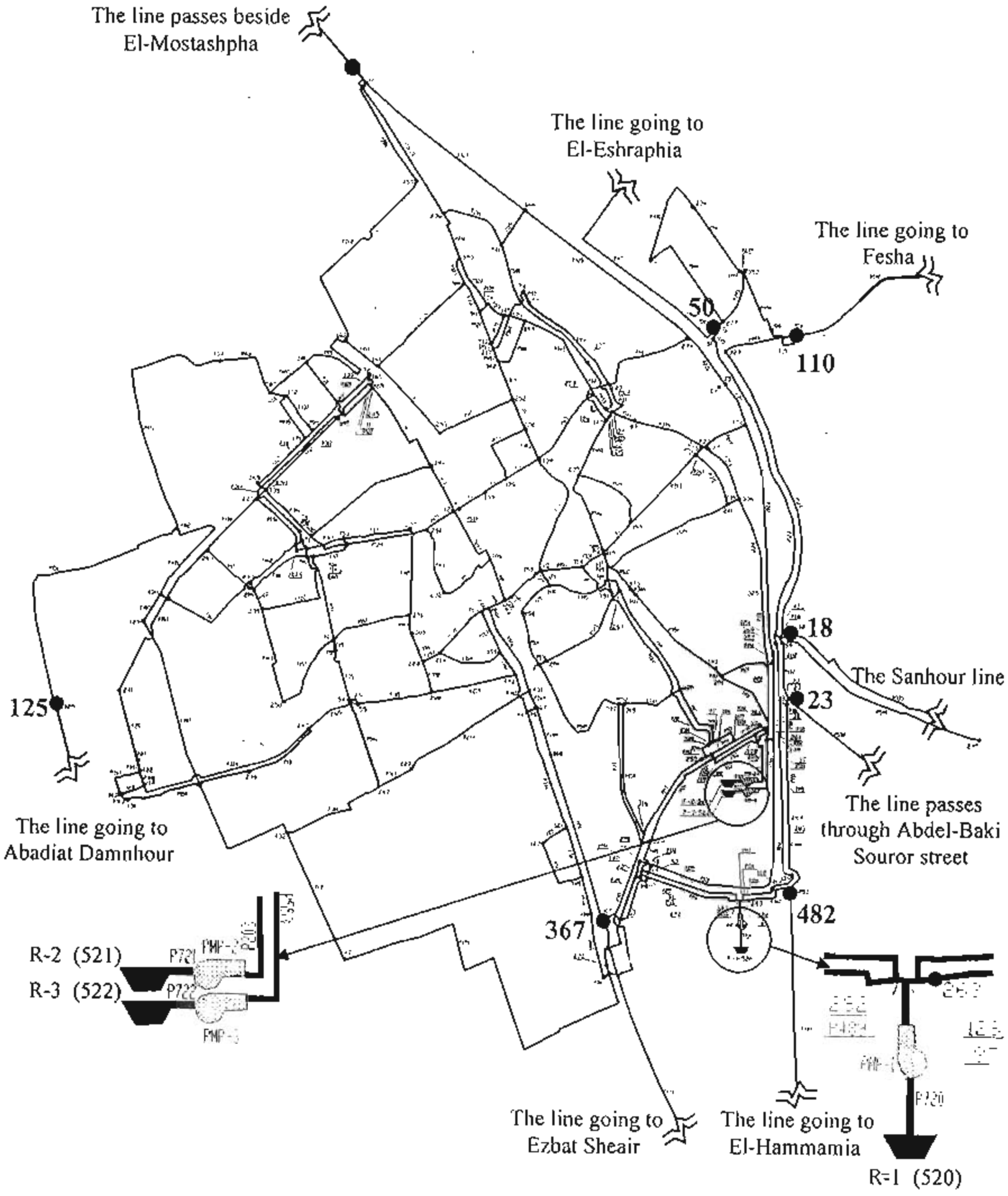

Figure (5): Layout of the Damnhour city water distribution network

Table (5): Results of measurements on the lines going to the rural network

Time	The line going to Abadiat Damnhour (Node 125)	The line passes through Abdel-Baki Souror street (Node 23)		The line going to El-Eshraphia (Node 50)		The line going to Ezbat Sheair (Node 367)	
	Q (l/s)	Q (l/s)	H (m)	Q (l/s)	H (m)	Q (l/s)	H (m)
12.00 p.m.	17.49	110.33	41.91	19.0	17.07	18.18	39.19
1.0	19.95	109.41	41.83	19.0	16.93	18.35	38.64
2.0	16.92	106.10	42.15	19.0	16.76	18.54	38.27
3.0	16.69	105.86	41.92	19.0	17.28	17.73	38.03
4.0	16.76	107.69	41.28	19.0	16.69	18.67	38.21
5.0	17.27	109.97	40.91	19.0	16.56	18.26	38.35
6.0	17.56	111.70	41.57	19.0	16.94	19.47	39.13
7.0	17.03	107.97	40.59	19.0	18.46	18.63	39.28
8.0	16.83	109.72	39.71	19.0	18.53	18.79	37.62
9.0	18.10	102.73	41.23	19.0	20.25	20.22	39.16
10.0	17.94	97.05	42.98	19.0	21.44	21.22	39.78
11.0	18.41	93.05	42.35	19.0	21.70	23.40	38.56
12.00 a.m.	17.84	83.20	38.17	19.0	21.51	22.09	38.27
1.0	13.96	66.09	35.76	19.0	22.98	21.75	37.66
2.0	14.88	---	---	19.0	19.46	22.81	24.48
3.0	16.08	---	---	19.0	20.76	23.15	27.93
4.0	16.53	---	---	19.0	21.05	21.25	29.17
5.0	16.85	---	---	19.0	21.37	22.14	29.26
6.0	17.13	---	---	19.0	25.02	22.16	29.18
7.0	20.19	92.24	38.41	19.0	25.46	22.21	35.90
8.0	---	99.93	39.21	19.0	22.73	23.67	39.30
9.0	19.3	102.89	39.94	---	---	21.15	38.65
10.0	18.65	109.76	40.82	19.0	19.67	21.29	39.88
11.0	17.58	111.66	41.8	19.0	20.50	18.23	39.46

Table (5): Results of measurements on the lines going to the rural network (continued)

Time	The line going to Fesha (Node 110)		The line going to El-Hammamia (Node 482)		The line passes beside El-Mostashpha	The Sanhour line (Node 18)	
	Q (l/s)	H (m)	Q (l/s)	H (m)	Q (l/s)	Q (l/s)	H (m)
12.00 p.m.	58.72	26.7	79.15	42.0	14.62	65.30	29.36
1.0	57.66	25.33	78.93	42.0	14.18	65.55	28.71
2.0	58.04	25.79	72.77	42.0	12.82	45.01	28.71
3.0	57.38	25.55	68.73	42.0	12.65	44.72	28.76
4.0	58.68	26.24	73.63	42.0	12.58	53.15	29.18
5.0	60.12	26.18	---	---	12.93	64.38	28.74
6.0	59.27	26.28	---	---	13.27	64.26	29.32
7.0	60.22	26.60	70.09	42.0	13.22	57.47	28.56
8.0	59.31	25.43	76.14	42.0	13.14	56.11	26.86
9.0	59.05	26.44	---	---	13.31	54.69	29.31
10.0	58.35	28.86	55.77	42.0	13.3	36.00	29.67
11.0	56.05	27.06	64.05	42.0	12.74	36.00	30.00
12.00 a.m.	55.35	27.38	52.12	35.0	12.76	25.31	29.62
1.0	53.69	28.18	63.23	35.0	10.57	---	---
2.0	38.13	21.86	59.54	35.0	9.74	---	---
3.0	36.98	21.72	51.93	35.0	9.29	---	---
4.0	30.4	20.31	48.82	35.0	9.15	---	---
5.0	34.49	21.18	49.44	35.0	9.21	---	---
6.0	37.4	19.91	58.61	35.0	11.53	---	---
7.0	56.67	26.78	74.73	35.0	---	37.71	26.01
8.0	59.9	27.49	72.02	33.0	---	69.55	30.99
9.0	60.35	26.70	38.29	35.0	14.3	71.81	29.74
10.0	60.17	26.97	76.57	35.0	14.26	61.08	29.29
11.0	60.38	26.94	79.02	35.0	15.31	87.94	32.29

4.2.1. Calibration and verification of Damnhour city network

Three important steps should be regarded, before using the mathematical model for water distribution networks in any hydraulic analysis, to gain confidence in the model results. These steps are calibration, verification and validation of the mathematical model. This sub-section highlights the calibration and verification of the model concerning Damnhour city network.

There are two defects associated with field data as follows:

- ✓ *Distribution of measurement points*: The measurement points should have a regular distribution throughout the network; this could not be verified or available in Damnhour city network. The available measurements are only at the connection between the rural network and the Damnhour city network.

- ✓ *Time of data collection*: All measurements should be performed simultaneously at all measurement points at the same time, also this is not verified in the Damnhour city network. Available measurements were carried out within a period of three months.

There are two water treatment stations supplying the network with water and a portion of this water going to the rural network through the eight pipelines connecting the city network with the rural network. In the mathematical model the two water treatment stations are expressed with three equivalent pumps, as shown in Figure (5). Equivalent pump (1) represents all operating pumps in the new Damnhour pump station while equivalent pumps (2) and (3) represent all operating pumps located at each of the two lines outside of the Czech pump station. It can be assumed that, all nodes in Damnhour network, other than connection nodes, have the same pattern and then there is only one group for all demand nodes.

The main purpose of this calibration problem is to predict both the main and secondary losses coefficients for each pipe through the network with the aid of the field data using the SO-OPTIM code, Table (5). The used field measurements in calibration are the average of the measured values of the maximum water consumption for Damnhour city network during the local time ranged from 3.0 p.m. to 4.0 p.m., Table (5).

Application of the proposed calibration model requires specification of upper and lower bounds on both the main and secondary losses coefficients. There are six types of pipe material, which exist in the Damnhour city network. These types are: Cast Iron (CI), Asbestos Cement (AC), Ductile Iron (DI), Glass Reinforced Plastic (GRP), Steel Type (ST), and Poly Vinyl Chloride (PVC). For the reason of absence of any information about the age of these pipes as well as the estimated roughness values for these materials are almost the same, value of roughness coefficient for each pipe can be selected between 80 and 140 during the calibration process. Also values of pipes secondary losses coefficients range between 5 and 50.

The following parameters are considered through the SO-OPTIM code: population size = 40; maximum number of generations = 500; crossover ratio = 0.7; mutation ratio = 0.1; the Idum number is taken (-1220); and uniform crossover is used.

Main and secondary losses coefficients for each pipe, in Damnhour city network, are obtained by minimization of the objective function, Eq. (14), under the hydraulic constraints. To verify the results of calibration, the relative absolute average errors in pressures, Eq. (15), is performed between the measured pressures and the corresponding simulated ones at the measurements nodes. These pressure values are listed in Table (6) under title of Run (1).

$$error = \frac{\sum_{i=1}^{I} \frac{|OP_{it} - PP_{it}|}{OP_{it}}}{I} \qquad (15)$$

in which, OP_{it} and PP_{it} = measured and simulated pressures at junction node i at time t and I = number of junction nodes with available pressure readings

The relative absolute average error is found to be 1.82 %. The values of both measured and predicted pressure readings are nearly the same except at node number (23), Table (6). The value of measured pressure head at this node, equal to 41.92 m, may not be correct because this pressure is nearly equal to the pressure head at new Damnhour pump station (42.0 m), Figure (5). Therefore, this value could be modified to become 39.83 m, which is the corresponding calculated one and closest to reality, Table (6). Another Run is performed with the modified value of pressure at node number (23). Results of calculated pressures are also shown in Table (6) under title Run (2). A comparison between the measured and calculated pressure values for this run shows the modified reading at node number (23) gives smaller relative absolute average error equal to 1.02 %. Therefore, any reading at measurement node number (23) greater than 41.0.m in Table (5) can be modified to become 39.83m. These modifications can improve the validation as shown in the coming sub-section.

Figure (6) shows the relationship between the values of used objective function, Eq. (14), and the number of generations. From this figure it can be seen that, the SO-OPTIM code gives a good solution after about 100 generations.

Table (6): Values of field pressure readings and corresponding predicted results for Run (1) and Run (2)

Measurement location	*Link No.*	*Node No.*	*Pressure head readings*			
			Run (1)		*Run (2)*	
			measured (m)	*Simulated (m)*	*measured (m)*	*Simulated (m)*
The Sanhour line	*P-335*	*18*	*28.76*	*28.71*	*28.76*	*29.14*
The line passes through Abdel-Baki Souror street	*P-550*	*23*	*41.92*	*39.83*	*39.83**	*39.65*
The line going to El-Eshraphia	*P-47*	*50*	*17.28*	*17.52*	*17.28*	*17.42*
The line going to Fesha	*P-502*	*110*	*25.55*	*25.69*	*25.55*	*25.82*
The line going to Ezbat Sheear	*P-199*	*367*	*38.03*	*38.77*	*38.03*	*38.29*
The line going to El-Hammamia	*P-409*	*482*	*42.00*	*41.23*	*42.00*	*41.22*

*This value has been modified by the authors

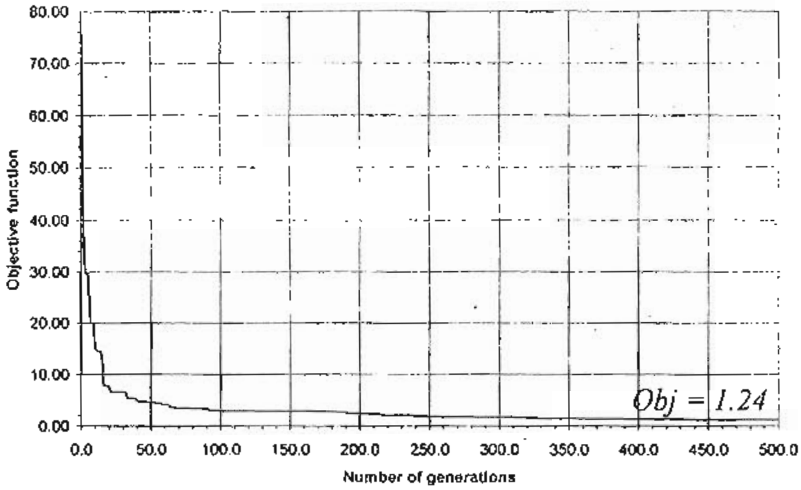

Figure (6): *Relationship between objective function, Eq. (14) and number of generations for Damnhour city network*

4.2.2. Validation of Damnhour city network

In this section the important third step (validation of a network) is performed. Validation means examining the model by computing the relative absolute average errors in pressures, Eq. (15), between field pressure readings obtained under different conditions (system demands, initial conditions, and operation rules) and the predicted ones. In this case study, the different conditions are both pattern multipliers and boundary conditions (outflows) at the eight separated points. To validate the model of Damnhour network, values of pressure at measurement stations are computed using the calibrated values of C_{HW} and K_s at each pipe in the network model by SFLOW code in periods; 12 p.m., 2 p.m., 4 p.m., 7 p.m. , and 9 p.m. Then, the relative absolute average errors in pressures are calculated for each of the previous mentioned time period. Table (7) shows values of both measured and simulated pressure readings and their corresponding relative absolute average errors for each time period. As shown from this table, the relative absolute average error for all the chosen time periods ranges from 1.3% to 2.86%, therefore these values could be accepted. The modified pressure head at node number (23) improves the validation of the mathematical model as mentioned before.

Figures (7) to (12) show the average measured pressures and the corresponding simulated ones for each time period at all measurements points. These figures are drawn for twelve hour period ranges between 12.0 p.m. and 12.0 a.m. instead of twenty four hours as usual. The reasons for this are the equivalent head-discharge curves for equivalent pumps are not available and some hydraulic devices were out of operation for some times at the rest of the day, Table (5). Good correlations between measured and simulated values are demonstrated in these figures. The relative absolute average error in pressures for each measurement node is calculated from Eq. (15). These errors could have acceptable values for all measurement points except at two measurement nodes (18) and (50).

For measurement point at node number (18), there are a big differences between the measured pressure heads and the corresponding simulated ones, about 3.0 m or more, at times (12, 1, 5, 6, 10, and 11 p.m.), Figure (7). The measured pressure heads at times (12, 1, 5, and 6 p.m.) approximate the measured pressure head at time 3.0 p.m. (calibration was performed at this time), as shown in Table (5), Nevertheless there is an increase of the outside flow at this node about 20 lit/sec. Any increasing in the discharge at any node is accompanied by a decrease in the head at that node if there is no change in the performance of pumps. An opposite trend can be noticed for times 10.0 p.m. and 11.0 p.m., a decrease in discharges than that used in calibration which are not suitable for the change (increase) in the measured pressure heads.

For node number (50), the measured flows are constant throughout the day, Table (5). The value of pressure at this node depends on the pressure at node number (18) as the two nodes are connected with three pipes (P336. P252, and P149), Figure (5). Therefore, any change in the discharge value for node (18) leads to change in pressure head value at this node and also noticeable changes in pressure heads at node number (50). This does not exist in reality as the pressure values were not simultaneously measured at these two nodes.

Figures (7) to (12) show a big difference (about 4.0 m) between the measured and simulated pressure readings at the time 11.0 p.m. This big difference may be due to measurement error at this time or all measurements were not taken simultaneously for the eight measurement points at this time (measurements throughout 3 months) or the available head-discharge curves for equivalent pumps are not suitable for this time. The last possibility is the closest to the real condition. However, results of validation could be accepted.

Table (7): Results of the Damnhour network validation

Time (hr)	*Node no.*	*Pressure head readings*		*Error, Eq. (6-3)*
		measured	*simulated*	
12 p.m.	*18*	*29.36*	*27.59*	*2.86 %*
	23	*39.83**	*40.30*	
	50	*17.07*	*18.24*	
	110	*26.72*	*26.93*	
	367	*39.19*	*39.99*	
	482	*42.00*	*41.88*	
2 p.m.	*18*	*28.71*	*28.99*	*1.3 %*
	23	*39.83**	*39.55*	
	50	*16.76*	*17.25*	
	110	*25.79*	*25.55*	
	367	*38.27*	*38.20*	
	482	*42.00*	*41.12*	
4 p.m.	*18*	*29.18*	*27.86*	*1.84 %*
	23	*39.83**	*39.57*	
	50	*16.69*	*16.66*	
	110	*26.24*	*25.41*	
	367	*38.21*	*38.42*	
	482	*42.00*	*41.16*	
7 p.m.	*18*	*28.56*	*27.80*	*2.48 %*
	23	*39.83**	*39.90*	
	50	*18.46*	*17.21*	
	110	*26.6*	*25.66*	
	367	*39.28*	*39.05*	
	482	*42.00*	*41.50*	
9 p.m.	*18*	*29.31*	*28.83*	*2.28 %*
	23	*39.83**	*40.34*	
	50	*20.25*	*18.45*	
	110	*26.44*	*26.64*	
	367	*39.16*	*39.45*	
	482	*42.00*	*41.85*	

* This value has been modified by the authors

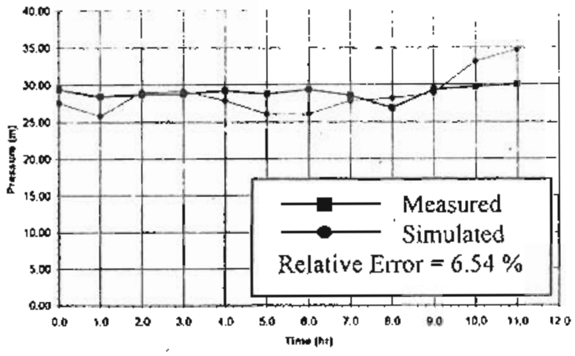

Figure (7): Simulation of pressure head at node (18)

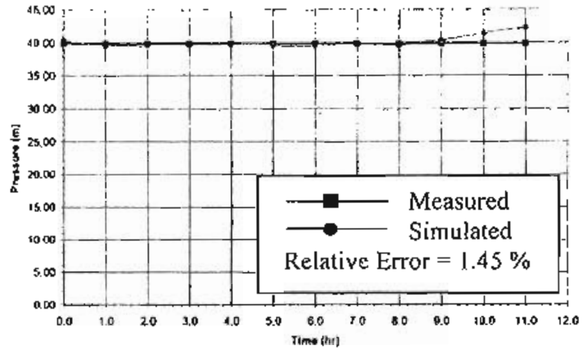

Figure (8: Simulation of pressure head at node (23)

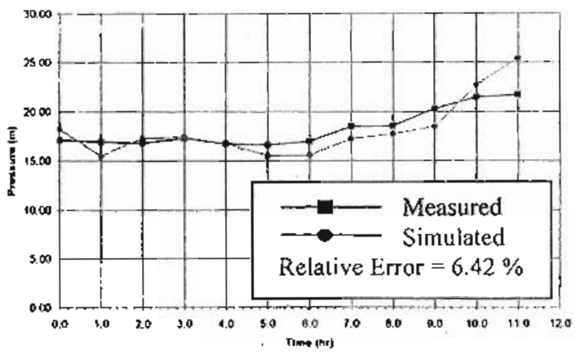

Figure (9): Simulation of pressure head at node (50)

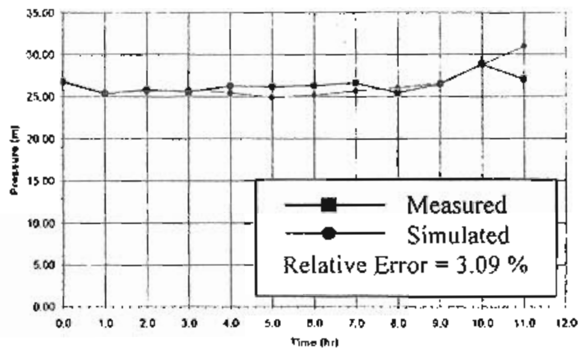

Figure (10): Simulation of pressure head at node (110)

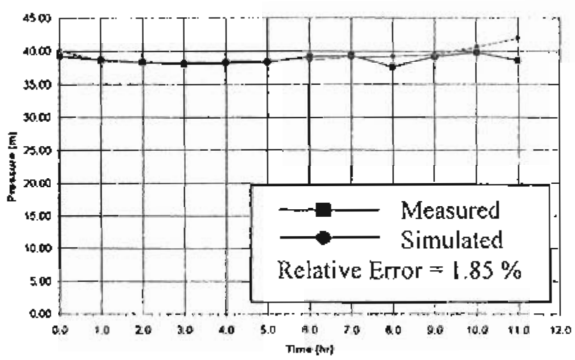

Figure (11): Simulation of pressure head at node (367)

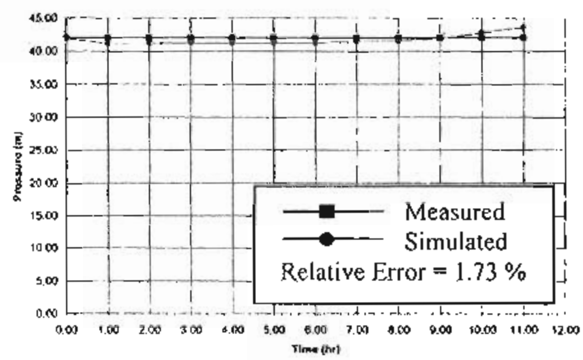

Figure (12): Simulation of pressure head at node (482)

5. CONCLUSIONS

The following conclusions could be drawn from this paper:

- A computer code with FORTRAN language named SO-OPTIM has been established to solve several problems in water distribution network such as: new design, rehabilitation, calibration, optimal operation of pumps, and optimal pressure regulation.
- SO-OPTIM applies both the principals of modified genetic algorithm optimization and the new method for pipe network analysis in steady state conditions.
- Capability of SO-OPTIM code to perform a calibration was illustrated in this paper by applying in two pipe networks. The first one is a hypothetical network with synthetic calibration data and the second one is a real water distribution network of Damnhour city.
- The calibration exercise included estimation of roughness coefficients and temporal demand distribution in the hypothetical network, in addition to the estimation of roughness and secondary losses coefficients for the Damnhour city network.
- The SO-OPTIM code has obtained a fairly good solution in less than 150 generations in the hypothetical network and in about 100 generations in the Damnhour city network.

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