DETERMINATION OF HYDRAULIC PARAMETERS OF WATER SUPPLY PIPE NETWORKS

by

K. Bozóky-Szeszich

Institute of Water Management and Hydraulic Engineering, Technical University, Budapest

(Received February 8, 1972)

Presented by Prof. I. V. Nagy

An extension of the existing water supply network or the reconstruction of an aged network may become necessary as the consequence of a rapid urban development, the growth of metropolitan area or the rise of water supply standards.

Extension or reconstruction should be preceeded by an investigation of the existing network area, with regard to the distribution of water demands and the capacity of the individual pipe branches, the latter in turn being inversely proportional to pipe resistance or wall roughness of the pipes concerned.

As well known, pipe wall is attacked by aggressive water and thus, the original smooth surface becomes rough. In addition, water containing calcium or iron will result in deposits on the pipe wall, constricting the cross section area and increasing pipe resistance.

The friction factor is generally calculated by aid of the Colebrook—White formula (also recommended by the International Water Supply Association):

$$\frac{1}{\sqrt{f}} = -2 \log \left(\frac{\varepsilon}{3.71\, D} + \frac{2.51}{Re \sqrt{f}} \right) \tag{1}$$

where ε is the pipe roughness in mm, D the pipe diameter in mm and Re the Reynolds number.

If the drop in pressure head h over a pipe length l is known together with the discharge Q or flow velocity v in the same pipe, then the friction factor f is to be calculated from the Darcy—Weisbach equation:

$$h = f \frac{v^2}{2g} \frac{l}{D} \tag{2}$$

followed by the calculation of the roughness ε from Eq. (1).

In case of used pipelines, however, the first question one has to face is the actual value of the diameter D. This problem arises primarily where a formation of deposits (incrustation) is likely to occur owing to the chemical ingredients of water. When repairing burst pipes, reduction in pipe cross section may be measured but water works seldom keep records on such measurements. Also, if there are such records, these may prove characteristic to the point of rupture only whereas other pipes of the same age may have an incrustation entirely different. This is why the *nominal diameter* has to be substituted usually into Eqs (1) and (2). This also means that in such cases it is the *apparent roughness* that becomes determined.

Obviously, it is sufficient to determine the roughness of the large-diameter pipes only, to be able to predict the hydraulic behaviour of the *whole network*.

The increased resistance of the major pipes may result in water supply troubles (pressure deficiency) over large areas, whereas the effect of small-size pipes is a local one only. As a matter of fact, even the smallest pipes play their part in conveying water but have a minor importance when compared to the larger ones. The only question left unanswered is the choice of a diameter below which the effect of pipes has to be neglected in comparison with the larger ones.

In principle, the roughness of major pipes can be determined by closing first all the connections along a certain reach and then, by producing over a short period a steady state of flow, measuring the discharge through the pipe reach and also the head at its both ends. Thus, all the data to calculate roughness are available.

There may be, however, a number of arguments against the applicability of this method. On the one hand, there are such *operational problems* as the admissibility of closing the connections or the possibility of closing reliably all the connections. Namely, if there are a good many connections to be closed and some of them are still tapping off water in this state too, then this may result in substantial errors of discharge determination.

Also the measurement of discharge is a difficult and expensive matter, except if provisions were made to this end right at the time the pipes had been laid. Consequently, there are other solutions to be looked for.

Theoretically, pressure head can be measured at any node of municipal pipe networks and thus, the drop in pressure head h is to be calculated. But the discharge Q between these nodes still remains unknown.

Water consumption appearing between two points of the pipe reach may be considered as concentrated at a half-by-half proportion to both ends [1] or in case of major consumers (e.g. a factory) a new node should be inserted.

If a network consists of w branches and k nodes, then one has to determine the following unknowns: k withdrawals at the nodes, w discharges in the individual branches and also w values of pipe roughness, totalling in $2w + k$ unknowns.

If pressure head is known in *all* the nodes, then w equations of the type

(2) may be established and so may be the continuity equation

$$\Sigma Q = 0 \tag{3}$$

for anyone of the nodes. The number of independent equations of the type (3) is $k - 1$, less than the number of unknowns, rendering thus the system of equations insolvable. On the other hand, if water withdrawal in B nodes is measured too and this measurement, together with the measurement of pressure heads, is repeated N times, corresponding to as many cases of operation, then the number of equations will be

$$N(w + k - 1)$$

and the number of unknowns:

$$N(w + k - B) + w\,.$$

(The roughness ε influencing the friction factor f will remain constant for all cases of operation.) Hence:

$$N = \frac{w}{B-1}\,. \tag{4}$$

Thus, in order to be able to determine the roughness of *all* pipe reaches and the withdrawals at *all* nodes (or, in other terms, the areal distribution of consumption), it becomes necessary to measure water withdrawal at *some* points, pressure head in *all* nodes and these measurements should be repeated N times in different conditions (various cases of pumping and consumption) with N being calculated from Eq. (4).

The performance of such a set of measurements is practically unthinkable, and thus, we have to abandon the idea of using the results of pressure head measurements to calculate the roughness and discharge in all pipe reaches of the network.

At this point the question may arise whether does one have to know the roughness of all pipe reaches? Maybe satisfactory results would be obtained by knowing the *average* roughness of the network or, by grouping the pipe reaches according to some points, by knowing the *average roughness of these groups.* (Grouping may be based upon age and pipe material, if there is one feeding point only. If there are more feeding points and different water qualities, then this latter circumstance should be paid special attention to.)

Fig. 1 gives a certain answer to the question put. By assuming a velocity $v = 1.0$ m/sec and a roughness $\varepsilon = 1.0$ mm differing from the real roughness, then the friction factor f_ε calculated from Eq. (1) and the friction factor $f_{1.0}$

pertaining to $\varepsilon = 1.0$ mm will show varying proportions. The figure shows e.g. that for $\varepsilon = 10$ mm and $D = 300$ mm $f_\varepsilon/f_{1.0} = 2.02$ which is a considerable discrepancy. But taking for instance $\varepsilon = 3.0$ mm, $f_\varepsilon/f_{1.0} = 1.045$. In other words, if the friction loss of a pipe having a roughness of $\varepsilon = 3.00$ mm is calculated as if it would have $\varepsilon = 1.0$ mm only, the error thus committed is 4.5 per cent only, which can be well tolerated.

Taking also some practical points of view into account, two further statements can be based upon Fig. 1:

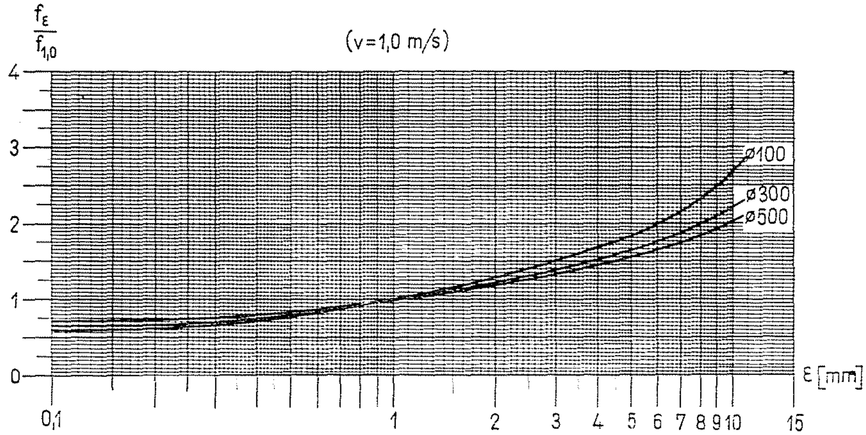

Fig. 1

a) it may suffice to determine the roughness of a reach *approximately* only (as in the above example, when tripling the value of ε, $f_\varepsilon/f_{1.0}$ will increase 1.045 times only);

b) if there is but a slight difference between the roughness of various pipe branches then — again looked upon from the point of engineering practice — it may well be sufficient to determine an average roughness.

Literature references contain a number of ways of determining average network roughness [2, 3, 4], and it was partly these examples that were followed when calculating the average roughness of a network or its roughnesses differentiated according to pipe groups.

The main point of the method lies in measuring the head at a few nodes of the network, obtaining for each of the nodes a measured head H_{mi}. Next, one assumes various values for the roughness ε, and to each ε there will pertain at each node a *calculated head* H_{ci}, and also a deviation $\Delta_i = H_{ci} - H_{mi}$.

The inverse of the functions $\Delta_i = f(\varepsilon)$ will yield at $\Delta = 0$ the value of ε, "felt" by the node. Furthermore, the minimum of $\Sigma|\Delta_i| = f(\varepsilon)$ or of $\Sigma\Delta_i^2 = f(\varepsilon)$ will yield the average roughness of the network.

It was investigated whether the above method suited to determine the average roughness and the roughness differentiated according to pipe groups. Calculations can only be performed in the knowledge of the areal distribution of consumption, which, however, can only be estimated. Therefore, investigations also were extended to find out the effect of uncertainties in consumption distribution upon computed results.

The value of calculated roughness is also affected by the diversity of the *actual* and the *calculated network*. In actual networks, even pipes with the smallest diameters are participating not only in distributing water but also in forwarding it. In calculated networks, however, small-diameter pipes of the network are partly or entirely neglected. One may ask how far this neglect would affect calculation results.

Methods to be used in performing measurements and calculations may depend to various extents upon the network size but experiences gained from earlier investigations may prove useful for smaller and larger networks as well.

In one of our investigations, referring to the network of a small Hungarian country town, *roughness* was *assumed* in various cases of consumption and heads in the nodes were *calculated*; taking some of these values *for measured ones*, roughness was calculated again.

The network investigated consisted of 175 pipe reaches and 125 nodes.

The simplified (calculated) network is shown in Fig. 3. The town is to be subdivided into 4 districts and an industrial area. The distributed consumption of residential districts and the concentrated consumption of the industrial plants are shown in Table 1. Consumption case 1 refers to daytime, cases 2, 3 and 4 to night time. The latter three differ insofar as it was assumed in cases 3 and 4 to withdraw 10 lit/sec each through three fire hydrants.

Table 1

District or node	Water consumption of residential areas and industrial plant, lit/sec			
	Case 1	Case 2	Case 3	Case 4
I	51.50	2.73	2.73	2.73
II	14.84	0.98	0.98	0.98
III	13.94	0.92	0.92	0.92
IV	3.67	0.24	0.24	0.24
80	13.33	0.92	0.92	0.92
78	3.33			
79	3.33			
27			10	
26			10	
39			10	
51				10
52				10
53				10
Total	103.94	5.79	35.79	35.79

When calculating heads in the nodes, the same distribution of consumption is assumed as when calculating roughness from head values H_{mi}.

In the case of actual pressure head measurements, however, *total consumption* is only known. Of course, there is no reason for not measuring withdrawal by major consumers (factories, hospitals) together with gauging the pressure. In such cases, the distribution of one part of the consumption becomes known and the remainder is to be distributed somehow over the network. (This distribution may be taken as proportional to the number of consumers, which, however, may prove utterly misleading. A better approach to actual conditions is obtained by distributing the remainder among the districts proportionally to their annual water consumption.)

In order to convey an idea upon the effect of distributing the consumption, the cases 11, 12, 13 and 14 were introduced. In cases 11 and 12, the *whole consumption* (including the concentrated withdrawals at nodes 78, 79 and 80) had been distributed uniformly over the network. In cases 13 and 14, consumption of the districts and that of node 80 have been distributed uniformly, but those in the nodes 27, 26, 39 and 51, 52, 53, respectively, were assumed as concentrated ones.

From among the investigations performed, two will be described below.

Network with uniform roughness

All pipes of the network shown in Fig. 2 were assumed to have a roughness $\varepsilon = 1.0$ mm, and subsequently, pressure head was determined on each node. The heads on 20 nodes being considered as pressure gauge readings, and assuming roughness values $\varepsilon = 0, 2, 4, 6, 8, 9$ mm, the values H_{ci} and Δ_i were calculated. The relationships $\Delta_i = f(\varepsilon)$ for the nodes 45 and 70 are shown on Figs 4 and 5, respectively, whilst the relationship $\Sigma |\Delta_i| = f(\varepsilon)$ is illustrated by Fig. 6. The last figure of curve labellings denotes the consumption case, the third figure indicates the number of roughness values assumed, which, in case of the now assumed uniform roughness, is necessarily equal to 1. Calculated roughness is indicated next to the curves. This has been determined by calculating first the coefficients of an *interpolation polynomial* from pairs of values $\varepsilon - \Delta$ or $\varepsilon - \Sigma |\Delta|$ and then, the calculated value of the roughness ε will be yielded by the vanishing point of Δ or the minimum of $\Sigma |\Delta|$.

Results of calculation are shown in Table 2, indicating the fact that except case 11, roughness has been obtained with a value fairly near to its real one. The latter case resulted in negative values of roughness at two nodes pertaining to $\Delta = 0$ from the relationship $\Delta_i = f(\varepsilon)$. This is physically impossible, but this erroneous result is by no means surprising, due to the distribution of consumption being made deliberately wrong (especially because

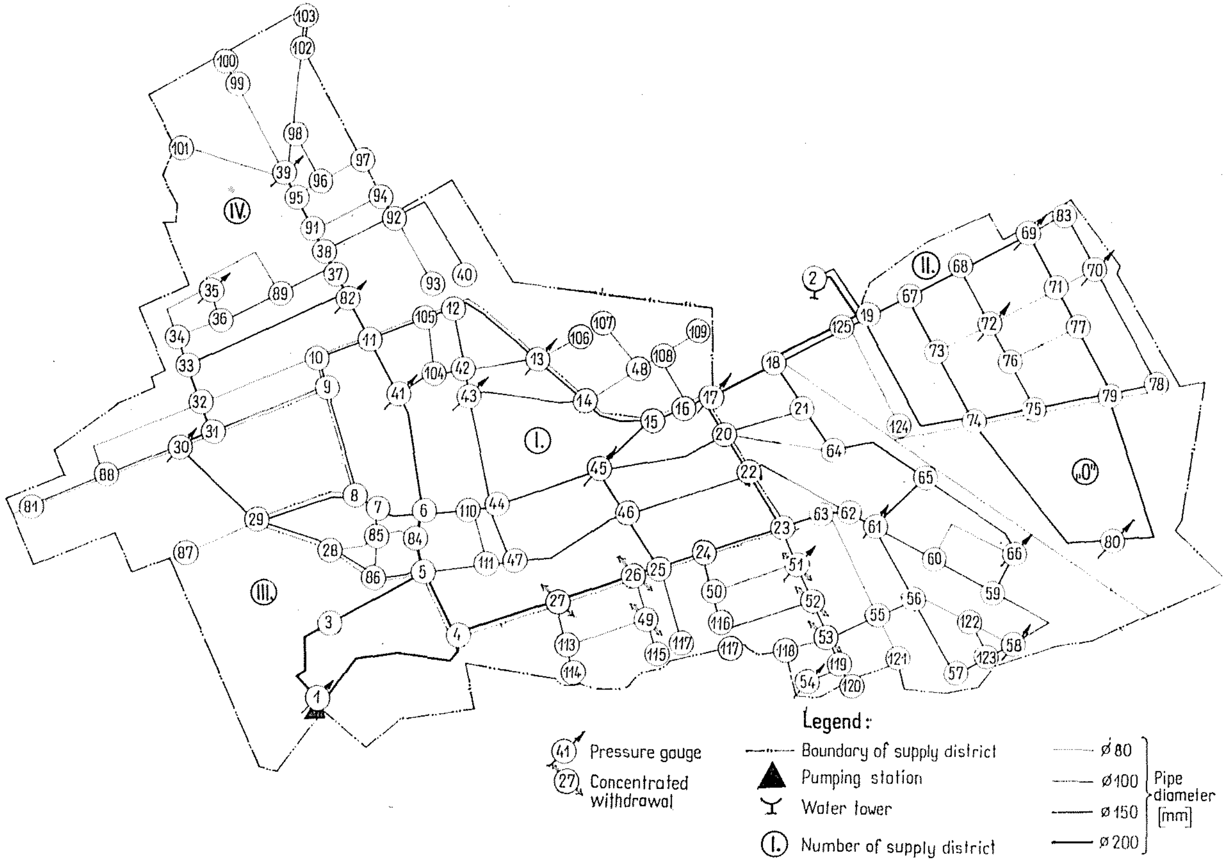
Legend:
Pressure gauge
Concentrated withdrawal
Boundary of supply district
Pumping station
Water tower
Number of supply district
ø 80
ø 100
ø 150
ø 200
Pipe diameter [mm]

Fig. 2

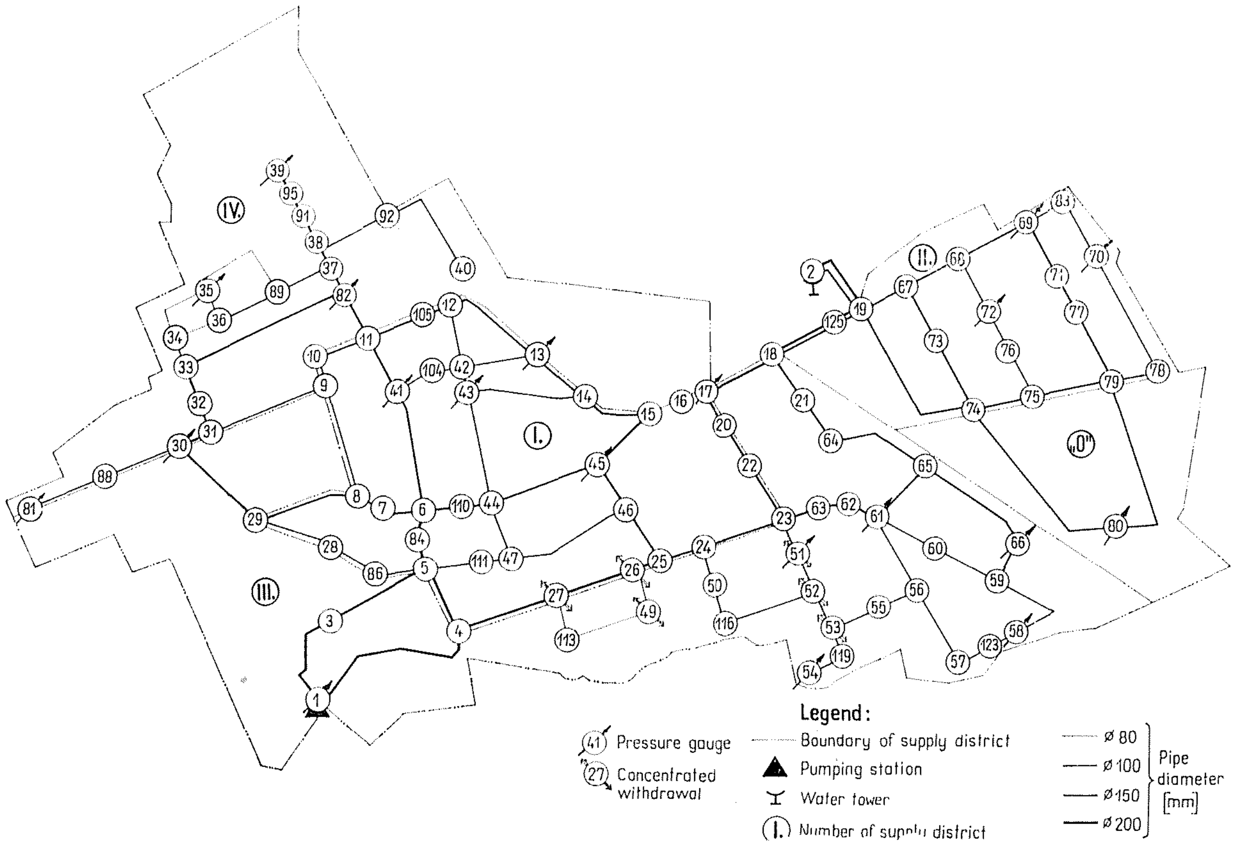
IV.
III.
I.
II.
„0"
Legend:
Pressure gauge
Concentrated withdrawal
Boundary of supply district
Pumping station
Water tower
Number of supply district
Ø 80
Ø 100
Ø 150
Ø 200
Pipe diameter [mm]

Fig. 3

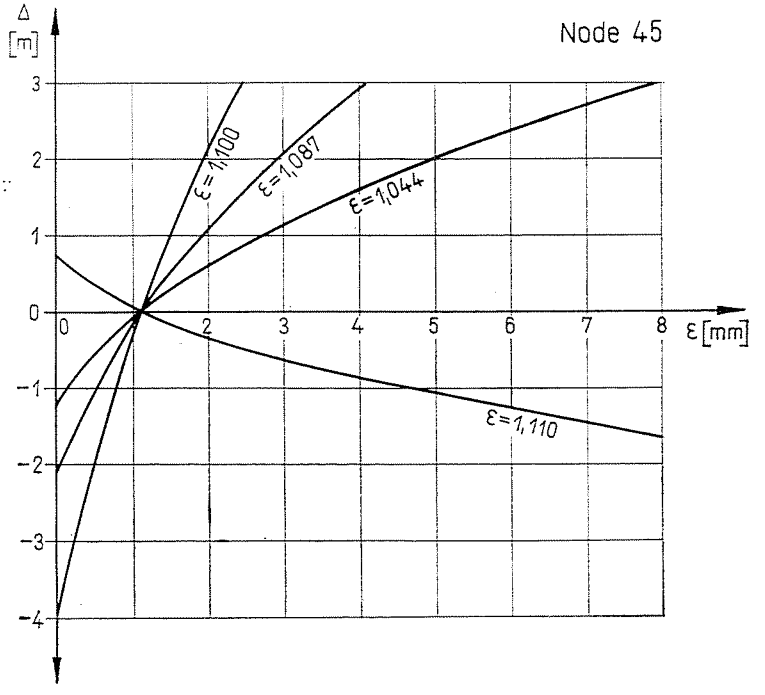
Δ
[m]
Node 45
ε=1,100
ε=1,087
ε=1,044
ε=1,110
ε [mm]

Fig. 4

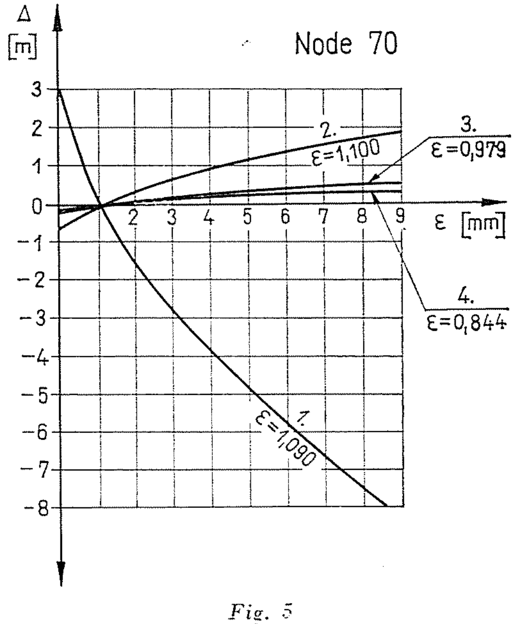
Δ
[m]
Node 70
2.
ε=1,100
3.
ε=0,979
4.
ε=0,844
1.
ε=1,090
ε [mm]

Fig. 5

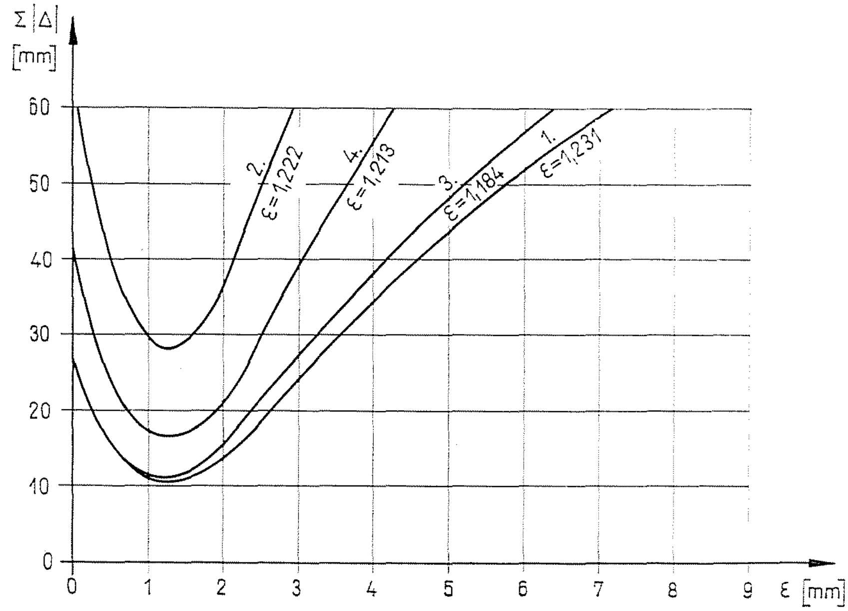

Fig. 6

Table 2

$\Delta_i = f(\varepsilon)$	Serial number of consumption case							
	1	2	3	4	11	12	13	14
	Calculated roughness in mm							
max	1.230	1.110	1.127	1.122	2.502	1.186	1.967	1.478
min	1.086	1.083	0.979	0.786	−0.442	1.107	1.002	1.004
mean	1.128	1.097	1.062	1.054	0.832	1.149	1.189	1.096
$\Sigma \lvert \Delta_i \rvert = f(\varepsilon)$	1.231	1.222	1.184	1.213	0.959	1.243	1.217	1.223
$\Sigma \Delta_i^2 = f(\varepsilon)$	1.335	1.320	1.282	1.317	0.901	1.351	1.325	1.327

of the measurable industrial consumption being uniformly distributed over the whole network). Such an effect should have appeared in case 12 too, but the wrong distribution of the low night-time consumption makes its effect less felt. Finally, in cases 13 and 14, the wrong distribution is counteracted by a concentrated withdraval.

One may ask why the departing values of $\varepsilon = 1.0$ mm have not been obtained in the cases 1, 2, 3 and 4 where consumption has been distributed correctly. There may be two reasons for that. First, the network was calculated by aid of the Cross method, and since the computer was programmed to stop at about $\Sigma h = 3.0$ cm, the condition for each loop $\Sigma h = 0$ was not satisfied. The second reason may be found in changing values of ε by too high steps when determining the relationship $\Delta_i = f(\varepsilon)$ or $\Sigma \lvert \Delta_i \rvert = f(\varepsilon)$.

The other question to be put is whether the use of the value $\Sigma |\Delta_i|$ or that of $\Sigma\Delta_i^2$ is more recommendable when investigating the nodes simultaneously. The latter one is counterindicated by the fact that errors produced by pressure gauges may have a highly disturbing effect upon the results. By the way, it is interesting to find that the equation $\Sigma\Delta_i^2 = f(\varepsilon)$ yields usually higher roughness values than $\Sigma |\Delta_i| = f(\varepsilon)$. The relationship $\Sigma\Delta_i = f(\varepsilon)$ should not be used, since the summation of magnitudes having opposite signs may falsify the results.

Deviations between the *real network* and the simplified one *underlying the calculations* is illustrated in Table 3.

Table 3

Values of calculated roughness for the actual and the calculation network, in mm

Item	Case 1		Case 2	
	actual network	calculated network	actual network	calculated network
Average of nodes	1.128	0.972	1.097	0.813
$\Sigma |\Delta_i| = f(\varepsilon)$	1.231	0.959	1.222	0.697
$\Sigma\Delta_i^2 = f(\varepsilon)$	1.335	0.901	1.320	0.679

The results are pointing towards the fact that roughness obtained from the calculated simplified network was lower than the ones obtained from calculating the actual network. This becomes obvious when one takes into account that the same consumption and pumping output were considered in both cases, however with different pipe diameters. Again, if there is a lower cross-section area, the same friction losses will occur at lower values of roughness. This may be seen well in case 2; whilst branches of smaller diameters have the primary task to *distribute* water in case 1, they also *convey* water in case 2.

Investigation of a network with non-uniform roughness

The branches of the network shown in Fig. 2 were supposed to have different roughnesses, with a distribution shown in Table 4.

Table 4

Assumed roughness of network branches

Pipe diameter (mm)	Roughness (mm)
80	$\varepsilon_1 = 1.50$
100	$\varepsilon_1 = 1.50$
150	$\varepsilon_2 = 0.50$
200	$\varepsilon_3 = 3.50$

In addition to roughnesses thus assumed, node heads were determined too. Some of these being handled as if they were pressure gauge readings the resulting roughness values have been looked for.

During the first stage of investigations the network was considered as one with *uniform roughness*, i.e., after having assumed the same value of roughness for all pipe branches the friction factors f were calculated and thence the average roughness ε_a.

The average roughness ε_a was calculated from consumption cases 1 and 2, as the average of the values yielded by these cases. The determination of roughness differentiated according to groups of branches will be shown in connection with case 1.

Investigations were carried out with the networks shown in Figs 2 and 3 as well as the one simplified still further, shown in Fig. 7. Values of average roughness are shown in Table 5.

Table 5

Calculated values of average roughness

Network shown in	ε_a (mm)
Fig. 2	1.77
Fig. 3	1.66
Fig. 7	1.20

In knowledge of the average roughness, the investigation into roughnesses differentiated according to groups of branches followed. This was performed by assuming an average roughness for the smaller pipes (80 and 100 mm diameter) whilst the roughness ε_2 of the 150-mm pipes and roughness ε_3 of the 200-mm pipes have been varied separately.

The curves $\Sigma|\Delta| = f(\varepsilon_2)$ calculated from the assumptions $\varepsilon_1 = 1.66$ and $\varepsilon_3 = \text{const}$ are shown in Fig. 8. The minimum of each of the curves $\varepsilon_3 = \text{const}$ can be determined; and from the polynomial passing these minima, the value of ε_3 can be determined too. Results are summarized in Table 6.

Table 6

Roughness values differentiated according to pipe branches

Network shown in	Roughness (mm)		
	ε_1	ε_2	ε_3
Fig. 2	1.77	0.62	3.58
Fig. 3	1.66	0.46	3.63
Fig. 7	1.20	0.38	2.69

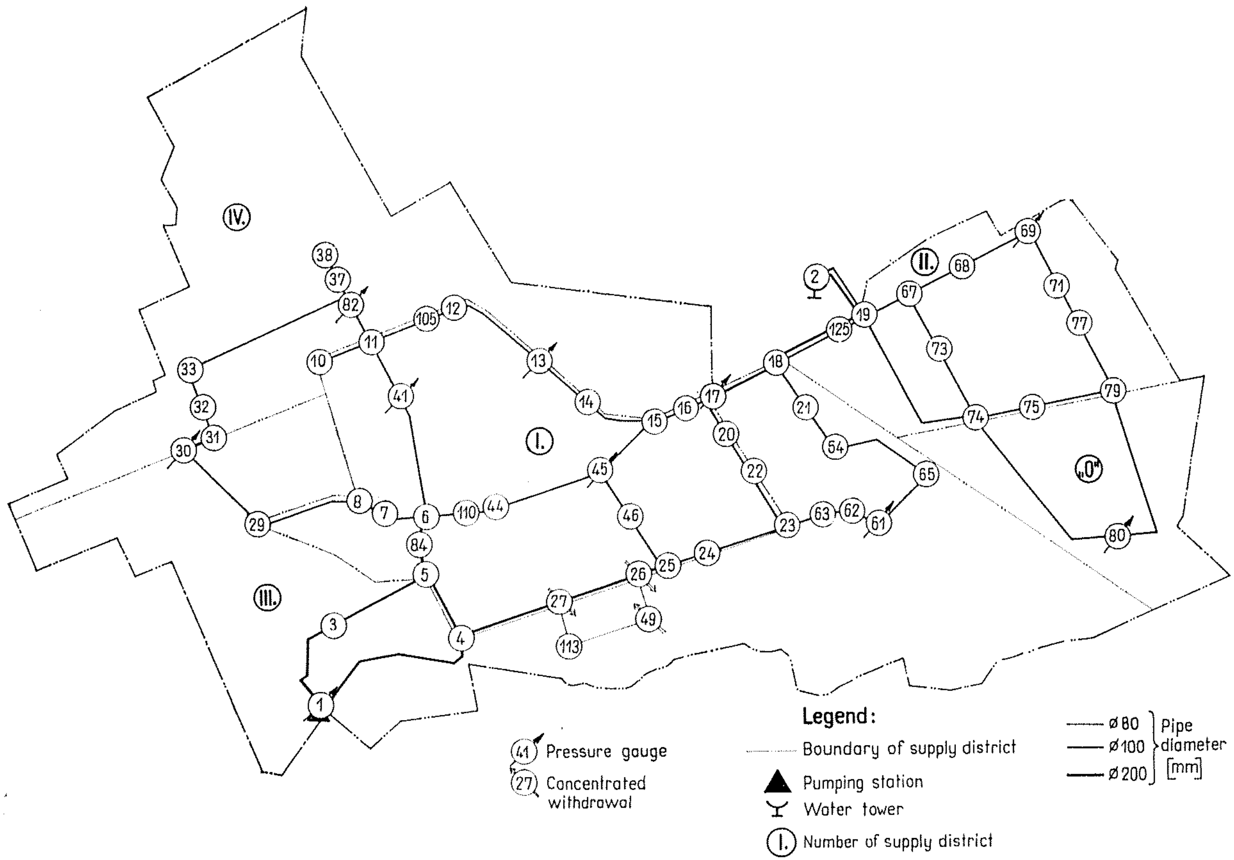
IV.
I.
II.
III.
„0"
Legend:
Pressure gauge
Concentrated withdrawal
Boundary of supply district
Pumping station
Water tower
Number of supply district
ø 80
ø 100
ø 200
Pipe diameter [mm]

Fig. 7

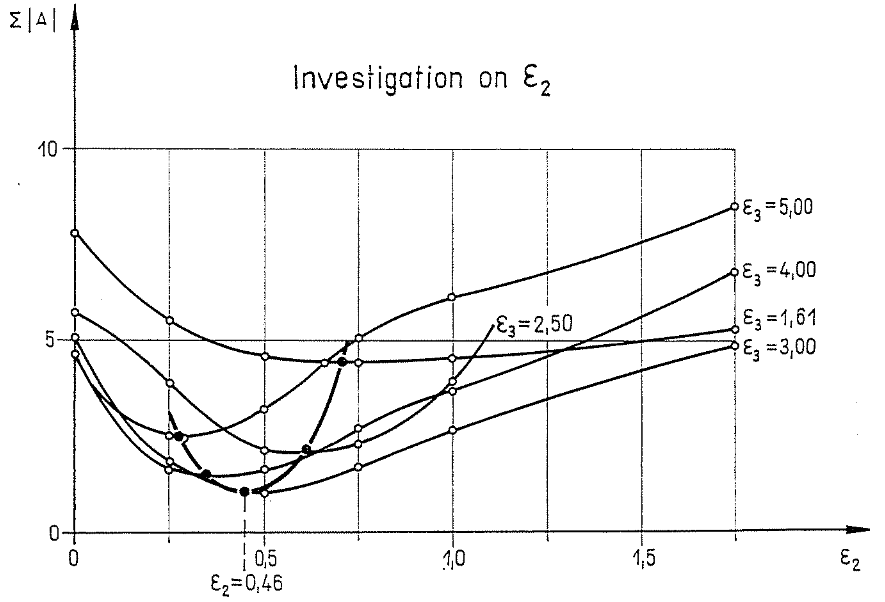

Fig. 8

Roughnesses determined for networks shown in Figs 2 and 3 are much more near to values of departure than are those calculated for the network in Fig. 7. These are lower values than expected for reasons already explained.

Conclusions

Investigations permit to draw the conclusion that a uniform roughness of the whole network is to be determined with an adequate accuracy, if

— the *areal distribution of consumption* is known,

— the *simplified calculation network* is not much differing from the actual network.

If the various groups of branches of the network have different roughnesses, then the average value as well as those differentiated according to branch groups can be determined with sufficient accuracy if conditions already discussed and those to be discussed below are fulfilled.

It was already asked whether a determination of the average roughness would suffice. In a way, this question is answered by Fig. 9; it shows the frequency of pressure head discrepancies for nodes of the network of Fig. 1 based upon the assumption of $\varepsilon_1 = 1.50$ mm, $\varepsilon_2 = 0.50$ mm and $\varepsilon_3 = 3.50$ mm. As to be seen, at about 80 nodes out of 125, the deviation is less than 0.5 m and the maximum deviation is 2.0 m. Such errors are admissible for practical purposes. Obviously, a number of similar investigations is needed to prove that it is sufficient to determine the average roughness.

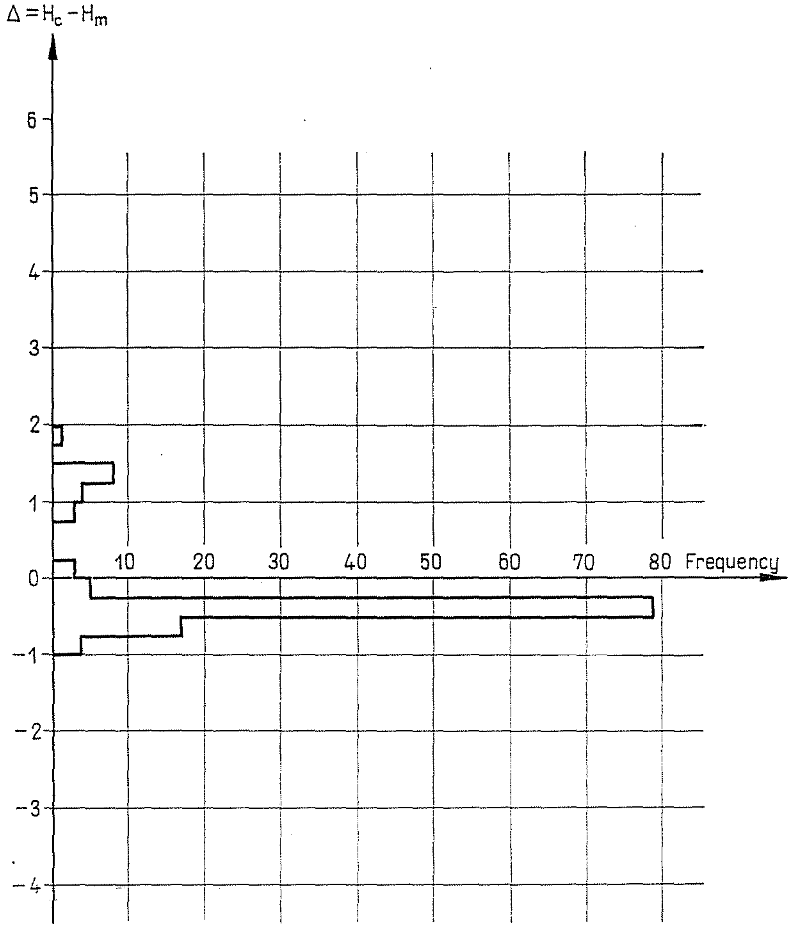

Fig. 9

The differences between simplified calculation network and actual network give rise to various problems. By examining several networks it will be certainly possible to determine a *correction factor* to be applied on the simplified network in order to convert its roughnesses into those of the real network. If roughnesses are investigated with the aim of being used for network extension or reconstruction purposes then the correction of calculated roughnesses can be omitted since the design of extension or reconstruction is also based upon the simplified network. An entirely different situation is met when one wants to collect information upon roughness changes by means of measurements repeated at certain time intervals. In such cases, correction cannot be avoided.

Calculation results are highly affected by the proper assumption of the areal distribution of consumption, the accuracy of pressure gauges and the proper selection of pressure gauging points.

Lowest errors in estimating the areal distribution of consumption are attained if measurements are performed during the night-time periods of low consumption with a simultaneous measurement of withdrawals by major consumers, like factories.

A drawback of night-time measurements, however, lies in the lower pressure drops between network nodes when compared with daytime operation, resulting in a more marked effect of errors committed in pressure head measurements. Another argument against night-time measurements is the fact that small pipes disregarded in calculations are distributing water primarily in

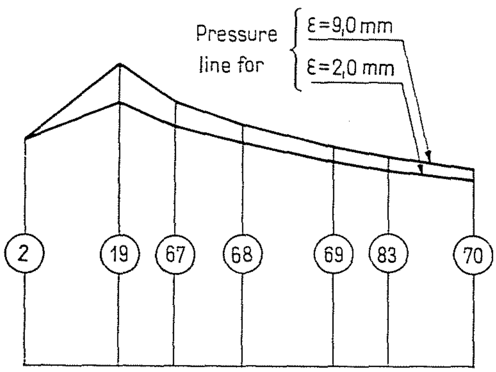

Fig. 10

daytime, but also carry water in night-time. This invariably results in lower roughness values of the calculated network in night-time than are those prevailing in the actual network.

Errors to be committed in assumptions on the areal distribution of consumption can be reduced in daytime too, if the consumption of major consumers is known and in addition, if there is a possibility of measuring the discharge in some of the larger pipes. The artificial withdrawal of water, as mentioned above, may prove efficient in lesser networks only.

The careful calibration of pressure gauges is absolutely necessary. Calibration should include the determination of the characteristic curve of the instruments, since the error pertaining to maximum deflection is insufficient to determine, owing to non-linearity of deflection with pressure changes.

The importance of the proper selection of pressure gauging sites is underlined by Fig. 5 where for cases 3 and 4, changes of roughness are but slightly followed by changes in Δ. (These two cases refer to night-time with a water tower at node 2 being in course of filling, and the whole phenomenon is explained by the sketch on Fig. 10.) If in any of the consumption cases Δ is varying feebly in function of the roughness, this may result in heavy errors of calculated roughness depending on errors committed in reading the pressure gauges. Thus it is expedient to carry out preliminary calculations for the sake of selecting gauging sites, directed toward the determination of the curves $\Delta i = f(\varepsilon)$; the steep limbs of these curves indicate the nodes suited as sites for pressure gauging.

Practical application

The above described method was used to determine roughnesses in the network of Kaposvár, in 1969. The simplified network had a length of 44 km, with 50% of the asbestos cement pipes being aged less than 20 years.

Measurements were made in night-time, using altogether 4 modes of operation. (The network is fed by 5 pumping stations, the discharge of which having been varied in order to produce modes of operation.)

Pressure head was gauged at 14 points. Roughnesses determined for the individual nodes ranged from 0.17 to 1.46 mm with an average of 0.52 mm. The functions $\Sigma |\Delta| = f(\varepsilon)$ and $\Sigma\Delta^2 = f(\varepsilon)$ have yielded a roughness $\varepsilon = 0.49$ mm

The values $\varepsilon = 0.52$ and 0.49 mm, respectively, to be regarded as average values are acceptable, since 50% of the network consists of asbestos cement pipes not too long in operation. On the other hand, the maximum value of $\varepsilon = 1.46$ mm seems to be low, because there are ferrous encrustations in a minor part of the netwoik.

Summary

The subject of this paper is the determination of pipe roughness in operating municipal water supply networks.

After having determined the pressure head in the nodes of the network by means of a simulated model, the average roughness of the network, or that differentiated according to branch groups is calculated. Attention is also paid to errors committed in assuming areal distribution of consumption and to possibilities of reducing these errors.

References

1. BOZÓKY-SZESZICH, K.: Some problems of hydraulic design of water supply networks. (In Hungarian). Hidrológiai Közlöny (Budapest) **3**, (1966).
2. KOTTMANN, A.: Die Berechnung von Wasserrohrnetzen auf elektronischen Rechenanlagen. Wasserfachliche Aussprachetagung des DVGW und VGW, Karlsruhe, 1967.
3. HOKE, G.—DÜRR, H. G.—APTE, H. H.: Die iterative Verhältnisrechnung für vermaschte Rohrnetze. Wasserfachliche Aussprachetagung des DVGW und VGW, Karlsruhe, 1967.
4. VIELHABER, H.: Ein Beitrag zur Untersuchung und Berechnung vermaschter Wasserversorgungsnetze. Dissertation. Technische Hochschule Aachen, 1966.

Senior Ass. Károly BOZÓKY-SZESZICH, 1111 Budapest, Műegyetem rkp. 3, Hungary