A COMPUTER PROGRAM FOR DESIGN OF SANITARY SEWER SYSTEMS

THESIS

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by

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ABSTRACT

A program is developed for the design of sanitary sewer networks when the sewer lines (links) connecting the manholes (nodes), the nodal elevations, the sewer lengths and a final outlet are specified. The program consists of two algorithms developed for hydraulic design and layout generation.

The hydraulic design algorithm is intended for design of networks with specified main or full layouts. The lift stations required may be sited either by the user or may be located automatically by the algorithm at nodes with depths exceeding a specified limit. Even concurrent assignment of both types of lift stations is within the capabilities of the algorithm.

The layout generation algorithm generates a main layout and transfers the data to the hydraulic design algorithm. Unlike model available in the literature, no initial layout is required for this algorithm. To generate the layout a standard Shortest Path algorithm is used to seek the path with the "shortest length" to the final outlet from every node in the network. When superposed, these

paths generate the routes from all initial manholes to the final outlet. The criteria tested to minimize the total excavation volume, which is found to be the most significant factor determining overall sewer system cost, included: horizontal and surface lengths of sewer links, natural slope and a hypothetical excavation volume (computed by assuming the upper cover depth and sewer slope to be minimum) for each sewer line. The relatively high computer storage requirement in generating a layout for large networks is overcomed by subzoning the network and then superposing the layouts generated for each subzone.

The layout generation and hydraulic design algorithms are applied together to sewer networks of different sizes, including PINARKENT (a resort town of 13,000), to test the various minimization criteria and hydraulic design parameters. The "hypothetical excavation volume" criterion is found to be quite efficient, especially in view of the fact that true optimality (or minimization) in excavation volume can only be achieved by the simultaneous solution of both the layout and the hydraulic design problems. As such, the program can allow for the easy evaluation of many alternative designs as well as the effect of various hydraulic design parameters. Hence, the objective of minimizing the dependence on engineering judgement in the overall design process has been achieved to a considerable degree.

ÖZET

Baca, kanal, baca kotları, kanal uzunlukları ve son toplama noktası belirli bir kanalizasyon şebekesinin tasarımını yapmak üzere bir bilgisayar programı geliştirilmiştir. Program hidrolik tasarım ve güzergah seçim işlemlerini yapan iki algoritmadan oluşur.

Hidrolik tasarım algoritması ana veya tüm güzergahı belirlenmiş şebekelerin tasarımını yapar. Gerekli terfi istasyonlarının yerleri kullanıcı tarafından belirtilebileceği gibi, derinliği belirlenmiş bir limiti aşan bacalara algoritma tarafından da otomatik olarak yerleştirilebilir. Hatta, her iki durumun birlikte ele alınabilmesi de mümkündür.

Güzergah seçim algoritması ana güzergahı belirleyip sonuçları hidrolik tasarım algoritmasına aktarır. Bu algoritma literatürde bulunan modellerin aksine önceden belirlenmiş bir şebeke gerektirmez. Her bacayı son toplama noktasına bağlıyan en kısa güzergahı bulmak üzere standard bir "En Kısa Yol" algoritması kullanılmıştır. Bu güzergahların birleştirilmesi ile bütün başlangıç bacalarını son bacaya birleştiren şebeke elde edilmiş olur. Toplam şebeke maliyetini belirleyen en etkin faktörün toplam kazı maliyeti olduğu bulunmuştur. Bu noktadan hareketle, yatay ve "yüzeysel" kanal uzunlukları, yüzeysel eğim ve her kanal için, kanal eğim ve üst baca derinliğinin minimumda olduğu kabulüyle hesaplanan hipotetik bir kazı, kriterleri toplam kazı hacmini minimize etmek üzere denenmiştir. Büyük şebekeler için gerekli büyük bilgisayar hafıza ihtiyacı şebekeyi daha küçük şebekelere bölüp sonra da her biri için belirlenmiş güzergahları toplıyarak önlenebilir.

Güzergah seçim ve hidrolik tasarım algoritmaları içlerinde PINARKENT'in de (13,000 nüfuslu bir sayfiye kasabası) bulunduğu değişik büyüklükteki şebekelere uygulanarak hem eniyileme (kazının minimize edilmesi) kriterleri hem de bazı hidrolik tasarım parametrelerinin etkileri incelenmiştir. Bu kriterlerden, hipotetik kazı kriteri, kazıda gerçek eniyilemenin ancak şebeke belirleme ve hidrolik tasarım problemlerinin birlikte çözülmesiyle elde edilebileceği de göz önüne alındığında, en başarılı ve hayli etkili bulunmuştur. Program bu haliyle, hem birçok seçenek şebekenin hem de çeşitli hidrolik tasarım parametrelerinin kolayca değerlendirilmesini sağlar. Böylece, başarılı tasarım için mühendislik tecrübe ve sağduyusuna olan bağlılığı azaltma yolundaki amaca büyük ölçüde ulaşılmıştır.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

A	Flow area, cm^2
B	Flow width, cm
C	Unit excavation cost
D, d	Pipe diameter, cm, m
DMIN	Minimum cover depth, m
D_1	Cover (nodal) depth of the upper node of a link, m
D_2	Cover (nodal) depth of the lower node of a link, m
K_Q	Average flow coefficient, m^3/day/m
L	Horizontal length of a link, m
L_R	Real surface length, m
M	Number of links
n	Manning's roughness coefficient
N	Number of nodes
Q	Discharge rate, cm^3/sec
Q_{ave}	Average wastewater discharge, m^3/day
Q_d	Design discharge, m^3/day, cm^3/sec
R_H	Hydraulic radius, cm
S	Sewer slope
SN	Natural slope
SMIN	Minimum pipe slope
V	Flow velocity, m/sec
V_E	Excavation volume, m^3
Y	Flow depth, cm
ΔZ	Elevation difference between two nodes of a link, m

Subscripts:

$_{max}$ denotes maximum

$_{min}$ denotes minimum

1. INTRODUCTION

The rapid growth of urban population has led to severe problems in management of urban wastewaters, consisting of domestic and industrial sewage as well as storm runoff. A sewer system is designed to remove unwanted liquid wates from an area. When the wates come from households and/or industries, the system is known as a sanitary sewer system. In case the wates originate from storm run-offs, the system would be known as a storm sewer. Large amounts of money and resources are involved in the design, construction, modification, operation and maintenance of urban sewerage systems.

There are municipalities which have combined sewer systems to handle both type of wastes together. Combined sewer systems are usually not preferred for modern sewer system design because of the following problems (Symons, 1967):

- The needs for collection of sanitary sewage are immediate, whereas those for storm run-offs are not.
- Conditions favor surface transmission of storm waters for long distances.

- Disposal of combined flow would generally require pumping, which would be unnecessary for storm runoffs.
- Treatment of storm waters is not necessary (since 1967, partial treatment is required - Winklehaus, 1977).

Even though many cities have extensive, old combined sewer systems, the trend during the recent years, has been toward construction of two separate systems. The main reason for this shift has been the increased necessity for construction of sewage treatment facilities to control pollution in streams and waterways. However, due to the above differences, the findings of the investigations dealing with storm sewer networks can find only limited use in design of sanitary sewer systems.

A sewer system is usually designed to serve a single drainage area and to operate entirely by gravity. It consists of a tree-shaped network of pipes (or links) connecting points of inflow (i.e., manholes). Such a sewer network may be defined by specifying the manholes and the connecting links. Then the size and slopes of all the pipes in the system are determined so that the construction and operational costs are minimized. Several studies dealing with the minimum-cost design of sewer systems have been reported in the literature (Mays and Yen, 1975; Tang, Mays and Yen, 1975).

The selection of a layout for a wastewater collection system is a complex task requiring a great deal of experience and engineering judgement. At best, the designer can investigate only a handful of possible layouts while seeking for the most suitable one. Despite the growing concern for the urban environment and the future expenses and effort involved, these conventional, intuitive design methods fail to account explicitly for the cost interactions of the various components of the system.

The large expenditures involved make it imperative that the most economical solutions be reached with the assistance of mathematical analysis. Specifically, a systematic approach must be sought so that such powerful tools as high speed digital computers can aid the engineer in the assessment of the alternatives. The results of the calculations, when combined with subsequent detailed investigations and sound engineering judgement, should yield routes superior to those determined by previous methods. Such an approach is sought herein.

In the present study the purpose is to develop a procedure and a program for design of sanitary sewer systems with particular emphasis on cost optimization of network layout. Given a drainage area, its topography and the layout, the design objective considered is to determine the diameter and invert elevations for each component of a sewer system. Finally, the new design program is applied to realistic examples to illustrate its advantages over the traditional design methods. A user's guide is prepared for the program and presented in the Appendix along with a worked example.

2. LITERATURE REVIEW

The work done on various aspects of the design of sewer systems is reviewed in this chapter. However, sewer systems will be described first to bring to attention the relevant characteristics.

2.1. SEWER SYSTEMS

Since the sanitary and storm sewers are to serve for different purposes, they are designed with respect to different criteria. Discussion of the design of storm sewers is beyond the scope of this study. Detailed information on design of storm sewers may be found in Yen and Sevuk (1975), Yen, et al. (1976), and ASCE (1969).

2.1.1. SANITARY SEWERS

Sanitary sewers, being specifically designed for domestic and/or industrial wates, must run along streets, alleys and right-of-ways to provide access to the adjacent properties and be placed at sufficient depths so sewage from the neighboring properties can flow by gravity, whenever possible. Depth of a sanitary sewer varies between a minimum cover depth and a maximum allowed depth. The minimum cover depth is required to protect the pipes from freezing and other external effects such as surface loadings. The maximum depth criteria is a factor which must be adapted to prevent enormous depths and excavation costs. Besides, presence of a rocky foundation or a high water table affects the working conditions adversely and increases both the construction and installation costs. On the other hand, infiltration of groundwater to existing sanitary sewers, causes a hydraulic overloading to both the collection system and the treatment plant. To avoid infiltration into sewers, either the sewer trench must be underdrained or cast iron pipes must be preferred.

Manholes are placed at major changes in grade, at pipe junctions, and at selected intervals to facilitate inspection and cleaning of the sewers. The layout of a sewer system can be represented by a network without considering the pumping stations and special appurtenances such as siphons, check valves, and junctions. Manholes connecting the pipes in a sewer network may be considered as the nodes of the system. Within this network of pipes and manholes, the sewage flow by gravity towards a specified outlet is preferred until the depth exceeds an allowable limit. When this maximum depth is exceeded a lift station must be built to elevate the wastewater up to the minimum cover depth to continue with gravity flow. The sewage outlet is normally a treatment plant. Occasionally it may be the connection to another existing sewer system or a receiving water body. There may be one or more such outlets depending on the size of the drainage area. As far as generation of a network layout is concerned, the lift stations would act as a local outlet. Thus a sewer network may have more than one outlet (multi-outlets).

Pipes ending at the same manhole may be connected at different elevations. However, it is necessary to set the elevation of the outlet pipe low enough to drain the deepest incoming pipe. Manholes in which an inlet pipe is much above the outlet pipe require a special exit section. Such manholes are known as drop manholes.

The discharge in a particular pipe is determined by several factors such as the population served by that pipe, the characteristics of that population and the amount of sewage inherited from other systems. The flow rate, along with the desired flow velocity, is then used to determine the required pipe size and its slope. The diameter of a pipe should normally not be smaller than the previous one. In determining the pipe size, special attention must be paid to legal requirements governing the minimum allowable pipe size as well as the available standard pipe sizes. Smaller sewers clog quickly and are harder to clean.

To prevent or reduce permanent deposition in sewers, a minimum permissible flow velocity at design discharge is specified. On the other hand, to prevent abbrasion of sewer materials due to high velocity flow, a maximum permissible flow velocity is also specified.

2.1.2. STORM SEWERS

The primary function of storm sewers is to prevent the inundation of streets, sidewalks, and other low-lying structures, together with disruption of traffic and damage to property. Wastewaters other than storm runoff are ordinarily negligible quantities in the hydraulic design of storm sewers. Therefore, storm sewers are designed to drain away the runoff of storms rapidly and without becoming surcharged. Surface runoff is led from street gutters into storm drains through street inlets and catch basins and through property drains. The quantity of storm water is 50 to 100 times that of sanitary sewage (Fair, Geyer and Okun, 1971). Therefore combined sewers are designed for the storm discharge rather than the sanitary flow rate. To induce the necessary cleaning velocities, sewers with special cross-sections may be used (Tekeli, 1982).

Sanitary and storm sewers show differences in the type of flow induced in each. In sanitary sewers, the discharge rates are low and relatively constant. Hence the hydraulic analysis of such flows may be simplified by assuming steady-flow conditions. In case of storm sewers, the unsteadiness of the flow control cannot be ignored. Yen and Sevuk (1975) showed that the hydraulic design of sewer systems becomes quite complex because of this unsteady nature of the flows and the mutual effects of the backwater in the sewers. Such factors are not significant in sanitary sewers and thus they are not considered in this study.

2.2. DESIGN OF SEWER SYSTEMS

The minimum-cost design of a sewer system can be considered in two phases: (1) optimization of the system layout, and (2) optimization of the pipe design parameters (elevations, slopes, diameters and manhole depths) for prescribed layouts. Due to the complexity of the problem when considering the layout and design simultaneously in an optimization procedure, usually only one phase of the problem is considered in any particular model. In this study, these effects will be discussed and summarized separately.

2.2.1. HYDRAULIC DESIGN MODELS

A review of the literature indicates that the problem of the hydraulic design of a predetermined layout is attacked by various investigators. The hydraulics of sewers, the quantity of sewage to be expected and the selection of pipe sizes are discussed in detail in ASCE (1969). Detailed discussions on hydrology and hydraulic design, environmental responsibility, economic considerations and the selection and installation of sewer systems for residental, commercial and industrial use are provided in AISI (1980).

Least cost design of sewer systems has been attempted by Holland (1966), Deininger (1969) and Gemmel (1972), using linear programming. However, these models are limited to linear cost functions or to nonlinear cost function that are separable for use in the linear programming algorithm. Meredith (1972), Mays and Yen (1975), and Mays and Wenzel (1976) suggest dynamic programming as an alternative technique which is flexible as far as the form of cost functions and constraint equations are concerned. Using discrete differential dynamic programming (DDDP) technique, Tang, Mays and Yen (1975) and Yen et al. (1976) developed models, which seek minimum cost by balancing installation costs and potential flood damages. All models pose computational difficulties for large networks.

2.2.2. SELECTION OF OPTIMAL LAYOUT

Models that select system layout include those of Liebman (1967) and Lowsley (1973). Here, optimal layout term refers to the layout requiring the minimum excavation and pipe costs.

Liebman (1967) developed a heuristic method to optimize the layout, assuming the pipe diameters to be fixed. Such an assumption is reasonable for small networks where a legal minimum size may be governing, but not for larger networks. In the method, the "best" layout is found by a search procedure. At each step, one link of the network is changed. The change is retained if the total cost decreases. This search is quite lengthy and the evaluation procedure requires excessive computation times. Liebman (1967) suggests use of the method for laying the trunk sewers without considering all of the branch lines. A trunk sewer (also called main line) receives wastewater from many tributary branches or laterals and serves a large area. Hence, this algorithm would be suitable for generation of layouts for small storm networks.

Lowsley (1973) presents an algorithm for obtaining an optimal layout within a range of pipe slopes. The algorithm begins with either a designer specified layout or a layout derived from Liebman's (1967) technique to find the longest path (or trunk) having the largest excavation and pipe cost. The algorithm connects each undrained node to the trunk by searching for the minimum branch cost. The efficiency of this algorithm is heavily dependent on the selection process for trunks and branches. Furthermore, pipe sizes are not specified and the flow characteristics are not considered. Thus the algorithm would be useful only for relatively small areas where a single pipe size would ordinarily be used. Lowsley's (1973) comparison indicated that Liebman's (1967) algorithm is significantly faster and usually obtains the optimal solution.

A dynamic programing model developed by Argaman, Shamir and Spirak (1973) attempted the simultaneous optimization of layout and design of sewer systems. Although dynamic programing enables complete freedom in selecting the objective functions, constraints, cost functions, etc., the applicability of this model is restricted to very small networks due to excessive computer memory and execution time requirements.

Mays, Wenzel and Liebman (1976) developed a heuristic optimization model for the simultaneous selection of layout and design of sewer systems. DDDP was used as the optimization technique but the solution was modified by introducing an Isonodal Line (INL) concept. These are imaginary lines connecting manholes that are located at ground surface elevations. The system depends on the INL construction and global optimum solutions cannot be guaranteed.

2.3. SUMMARY OF LITERATURE REVIEW

A review of the literature clearly indicates that the problem of optimality in determining the layout and hydraulic design of a sewer system has not been solved satisfactorily. Most solution methods are limited to small areas due to their computer memory and Central Processing Unit (CPU) time requirements; hence, they are not suitable for practical use. Finally, some are only heuristic procedures which form a step towards the development of a precise solution to the sewer design problem.

The present study attempts to develop a procedure and a program which will avoid the need for excessive computer memory and CPU time, for the design of sanitary sewer networks. Particular emphasis is placed on the optimization of the system layout. The objective function to be minimized is the total cost of the sewer system.

3. DESIGN OF SANITARY SEWER SYSTEMS

Within the framework of the discusssions presented in the previous sections, a program (SEWNET) is developed for design of sanitary sewer networks. The program consists of two algorithms: hydraulic design and layout generation as can be seen in the flowchart presented in Figure 1. The hydraulic design algorithm is intended for networks with specified layouts. The description of the layout may be possible either by specifying the flow directions in a full network or in a main layout. Here, main layout term refers to a layout which drains all the nodes, except the outlet, in a network. So it contains N-1 links, where N is the number of nodes in the network. For networks without a given layout, the layout generation portion of the program first generates a main layout, connects the unused links and then completes the hydraulic design. A flag variable (MLAYOT), provided by the user identifies the computational scheme to be followed (Figure 1). The development as well as the details of the algorithm is described in this chapter along with a discussion of the relevant design criteria.

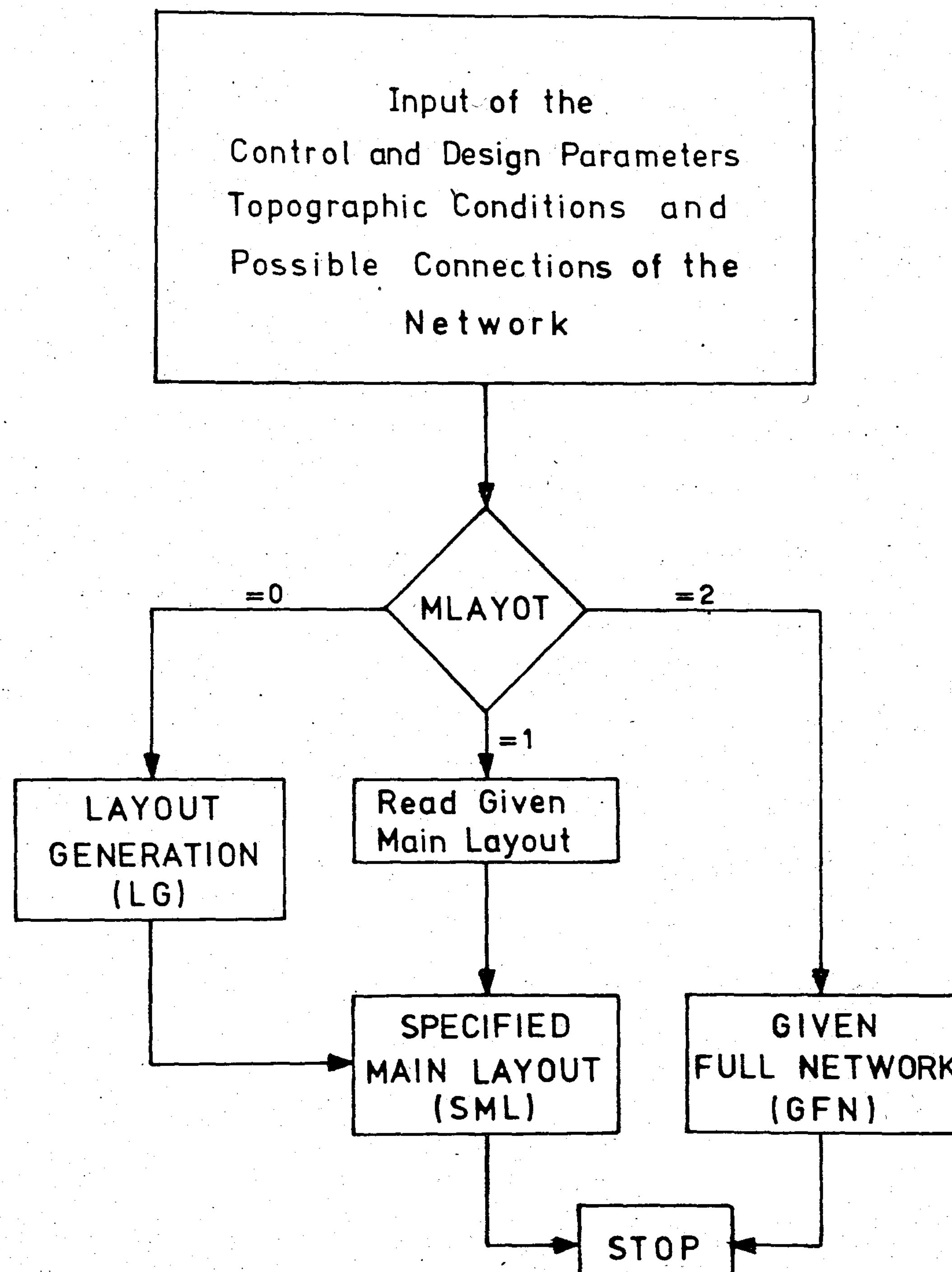


FIGURE 1. A Program for Design of Sanitary Sewers (SEWNET).

3.1. HYDRAULIC DESIGN OF SANITARY SEWER NETWORKS

In this section, the design criteria and computational scheme is presented first. Then, the algorithm developed for the hydraulic design of the systems is presented.

3.1.1. DESIGN CRITERIA

Once installed, it is difficult and expensive to expand or to increase the capacity of a sewer system. Therefore, it is common practice to take precautions to maintain a continuous operation. American and Turkish standards are discussed comparatively.

3.1.1.1. Quantity of Sanitary Sewage

Separate sanitary sewers are provided primarily to carry the domestic and industrial wastes of a community. So, connection of roof, yard and foundation drains to the sanitary sewers should be prohibited. However, water leakage into the sewers (infiltration) is always a possibility due to cracked pipes, defective joints, faulty manholes and/or improper house connections. Due to this inevitable addition to the total flow, sanitary sewer design quantities must include an allowance for non-waste components.

Design period during which a sewer system will serve must be decided upon prior to the design of the sewer systems. Then consideration must be given to the quantity of wastewater, which is largely a function of the population served, population density and water consumption, to be handled. Iller Bankası (1972a) suggests a design period of 30 years for sanitary sewers. But a rapidly growing population may make the use of a long design period uneconomical. After determination of the design period, population can be estimated using the suggested methods in Iller Bankası (1972a).

The average daily per capita domestic wastewater flow used for design purposes may be determined as 70% of the daily per capita water consumption (ASCE, 1969; Muslu, 1974). In addition, the contribution from touristic, commercial, industrial and other facilities and infiltration from ground water must be considered in determination of average wastewater discharge (Q_{ave},m^3/day).

Infiltration rates vary, depending on sewer construction, type of soil, elevation of water table, manhole leakage, roof drainage, etc. However, when making estimates for design purposes, the sewer layout, size and depths are not known; therefore, estimates based on values reported in literature may be used: 5000 gpd per mile (13.75×10^{-5} m^3/sec per km), or 200 gpd per acre (2.16×10^{-5} m^3/sec per hectare) or on a per capita basis values between 10 and

40 gpcd ($4.38\text{-}17.52 \times 10^{-7}$ m^3/sec) are often used (Symons, 1967).

In the design of sanitary sewers and treatment works where wastewater contributions govern, the daily minimum, maximum and mean discharge rates are important. The mean daily flow of sewage is derived from the analysis described above. Then the design discharge of sewer system is determined as (ASCE, 1969; Muslu, 1974):

$$Q_d = 1.5 * Q_{ave} \tag{1}$$

The daily minimum and maximum discharges are needed in determining treatment plant capacities and sufficiency of the flow velocities, pipe sizes and slopes. For these ASCE Manual (1969) and Muslu (1974) suggest:

$$Q_{max} = 2.25 * Q_{ave} \tag{2}$$

$$Q_{min} = \frac{2}{3} * Q_{ave} \tag{3}$$

These computations should be prepared for all regions of a drainage area where discharge factors differ significantly. In case of homogeneous regions, for ease in computation an average flow coefficient (K_Q) may be as:

$$K_Q = \frac{Q_{ave}}{\Sigma \text{ pipe length}} \ (m^3/day/m) \tag{4}$$

This coefficient can be increased with a weight factor for densely-populated regions. Multiplication of the length of a pipe with its flow coefficient gives directly the average discharge for that pipe.

3.1.1.2. Depth of Sewer

Insofar as feasible, sewers should be laid at sufficient depths to receive the contributed flows by gravity. Unjustified costs may preclude the lowering of a whole sewer system to provide service for only a few houses. Such cases may require individual pumping facilities. Normally house connections are laid at a slope of 2 percent (ASCE, 1969).

Sewers must be placed at depths that will not be susceptible to frost and allow for sufficient cushioning to prevent breakage due to ground surface loading. Therefore, minimum cover depths must be specified. Fair, Geyer and Okun (1971) state that a cover depth of 60-90cm should be adequate to prevent occurrence of such failures. Iller Bankası (1972b) suggests different cover depths depending on the altitude of the drainage area.

On the other hand, large sewer depths should also be avoided due to the increased possibility of encountering rocky soil formations and difficult working conditions, both of which increase construction costs. Also, on a flat drainage area a deep manhole will lead to greater depths at the consecutive manholes. However, ASCE (1969) points out that sewers as deep as 3.6 m or more may not be uncommon in business or commercial districts. This is allowed to accomodate the underground facilities normally found in such areas.

3.1.1.3. Minimum and Maximum Velocities

A sanitary sewer has two main functions: to carry the discharge for which it is designed and to transport the suspended solids so that deposits in the sewers are avoided. Therefore, it is essential that the sewers have adequate capacity for peak flows and they function at minimum flows.

Minimum velocities should be selected so as to prevent deposition and to prevent or to retard sulfide formation. Commonly, slopes are calculated so that when flowing half-full or full, the velocity will be 0.6 m/sec for sanitary sewers or 0.9 m/sec where sand and gravel exists (ASCE, 1969). Iller Bankası (1972a) requires a minimum velocity of 0.5 m/sec. In addition, a minimum flow depth of 2 cm is also required to prevent critical situations in view of deposition.

Slope calculations are based on the assumption that these minimum slopes will produce self-cleaning velocities.

However, upper reaches of sanitary sewers generally have shallow flow depths. Since the pipe size employed is relatively large whereas the collected wastewater is relatively small, self-cleaning velocities cannot be attained in such sewers. These sewers must be flushed out from time to time by providing flushing manholes at the beginning of such lines.

Velocities in excess of 10 m/sec have been found harmful to concrete channels, due to abrasion. So, a limiting velocity of 3 m/sec is often taken, to prevent occurrence of scour and other undesirable effects of high velocities.

In the ideal case the velocity of flow in all pipes of a sewer system should be within the following range:

$$Q_{max} \rightarrow V < V_{max}$$
$$Q_{min} \rightarrow V > V_{min}$$

But this may be a short range to achieve and sometimes may only be possible by providing relatively steep pipe slopes due to assignment of a minimum sewer size. So, in practice exceeding the minimum velocity at the maximum discharge is considered as satisfactory (Tekeli, 1982):

$$Q_{max} \rightarrow V > V_{min}$$

This maintains the self-flushing of each pipe once a day.

3.1.1.4. Pipe Diameters

The design diameter of a sewer pipe is the smallest commercially available size that has a flow capacity equal to or greater than the design discharge. The sewers should be no smaller than 8 in. (ASCE, 1969) or 20 cm (Iller Bankası, 1972a) in diameter to prevent clogging.

A 20 cm diameter sewer pipe must be laid at a slope of 1/300 to induce a flow velocity of 0.6 m/sec. The slope which induces the minimum velocity at the minimum sewer size is referred to as the minimum slope.

The minimum flow depth and velocity requirements are not so crucial for storm sewers, since they are designed to flow full. To ventilate the sewage, sanitary sewers are designed to flow at 40-80% of full capacity. Ventilation is necessary to avoid excessive oxygen deficiencies, which induces a septic condition leading to sulfide production. Manholes and building vents help to keep sewers sufficiently ventilated.

A final note about the sewer size is: at any junction or manhole, a downstream sewer cannot be smaller than any of the upstream sewers at that junction. However, Yen and Sevuk (1975) have shown that this is valid only for adjacent links having the same slope. Hence, if the slope of a downstream sewer is high enough to carry the incoming discharge within the specified velocity constraints, then this restriction may be removed.

3.1.1.5. Summary of Constraints

TABLE 1. TABLE OF CONSTRAINTS

Constraints	American Practice ASCE (1969); Fair, Geyer and Okun (1970)	Turkish Practice İller Bankası (1972 a,b)
Minimum Cover Depth	0.60 - 0.90 m	≈ 1.00 m (Changing with Altitude)
Minimum Velocity	0.60 m/sec	0.50 m/sec
Maximum Velocity	3.00 m/sec	3.00 m/sec
Minimum Flow Depth	—	2 cm
Minimum Sewer Size	8 in.	20 cm

As evident in Table 1, Turkish and American specifications show little differences. To increase the flexibility of the developed algorithm the constraints showing differences are considered as input variables while the rest are declared in the program.

3.1.2. FLOW COMPUTATIONS

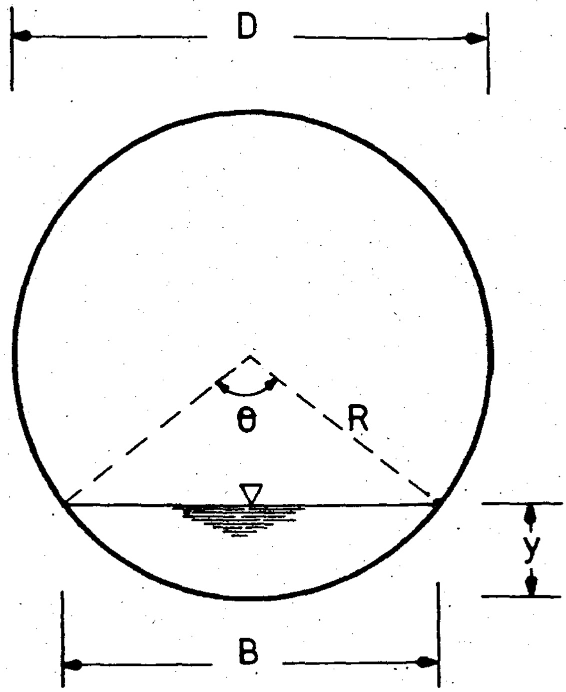
The design of sanitary sewers is concerned with the hydraulic performance of partly-full and full sections. Partly full pipe flows are computed using either Kutter's or Manning's formula. Both formulas give approximately the same results in usage of alignment charts for solution. But Manning's formula, because of its greater simplicity in specifying channel roughness, has replaced Kutter's formula in computerized engineering practice.

Using Manning's formula, flow and hydraulic elements in a partly-full circular section can be computed in the following iterative way. Here, Figure 2 gives the relationships for the hydraulic elements of a partly-full circular section with a known diameter.

For steady flows Manning's equation can be written as:

$$Q = \frac{4.642}{n} A \, R_H^{2/3} \, S^{1/2} \qquad (5)$$

where Q = discharge, cm^3/sec; n = Manning's roughness coefficient; A = flow area, cm^2; R_H = hydraulic radius, cm; and S = sewer slope. Substituting the flow area, A, and hydraulic radius, R_H, given in Figure 2, into the above equation and solving for Θ yields:



Flow Area : $$A = \frac{D^2}{8}(\theta - \sin\theta)$$

Hydraulic Radius : $$R_H = \frac{D}{4}\left(1 - \frac{\sin\theta}{\theta}\right)$$

Flow Depth : $$y = \frac{D}{2}\left(1 - \cos\frac{\theta}{2}\right)$$

Flow Width : $$B = D\sin\frac{\theta}{2}$$

FIGURE 2. Hydraulic Elements of a Partly-full Circular Sewer (Θ in Radians).

$$\Theta = \sin\Theta + \left\{ (2/R) \left[\frac{Q \cdot n}{4.642 \; R \cdot S^{\frac{1}{2}}} \right]^{0.6} \right\} \Theta^{0.4} \qquad (6)$$

where R is the sewer radius in cm. This equation can be solved for Θ iteratively after determining the design diameter. The design diameter should satisfy all the appropriate constraints and have a flow capacity equal to or greater than the design discharge. The minimum required sewer diameter D, can be computed from Manning's formula written for full pipes:

$$D = (0.691 \frac{n}{\sqrt{S}} Q_d)^{3/8} \qquad (7)$$

in which D is in cm and the design discharge, Q_d in cm^3/sec.

Then an estimated Θ value (Θ') is substituted into the right hand side of Equation 6 to obtain a new Θ value. This new Θ is taken as Θ' and the iteration process is continued until a percentage error, defined as $\varepsilon = \frac{\Theta - \Theta'}{\Theta}$, is reduced below 0.001 (Croley, 1977). Finally, the Θ value found at the end of the iteration process is substituted into the expressions in Figure 2 to determine the flow area, velocity and depth.

3.1.3. AN ALGORITHM FOR HYDRAULIC DESIGN

The hydraulic computations for a sanitary sewer network with a prescribed layout was presented in the previous sections. These computations, including the iterative computation scheme for each pipe, are routine, but tedious. A computer program, developed by Alper et al. (1980) and described by Tekeli (1981), for the computation and tabular printing of hydraulic and topographic elements of a sanitary sewer network under the declared constraints, is used. This algorithm is modified here to increase its adaptability to do the following: adjustment of original data for multi-outlet declaration by Subroutine ADJUST, assignment of lift stations by Subroutine POMPA to nodes with depths exceeding the allowable limit, and the hydraulic design of a network in which all the flow directions are specified by Subroutine FULNET. When more than one outlet is (multi-outlet) is declared, the last one in the list is the final outlet while the others are lift stations.

Here, the hydraulic design algorithm for a specified main layout will be presented in detail. However, for a given full network, to avoid repetition, only the differences from the previous algorithm will be discussed.

3.1.3.1. Design of a Specified Main Layout

The procedure used for the hydraulic design of a sanitary sewer network with a specified main layout is illustrated in the flow chart in Figure 3. The program can handle both the given and generated main layouts. The computations carried out in each of the steps is explained

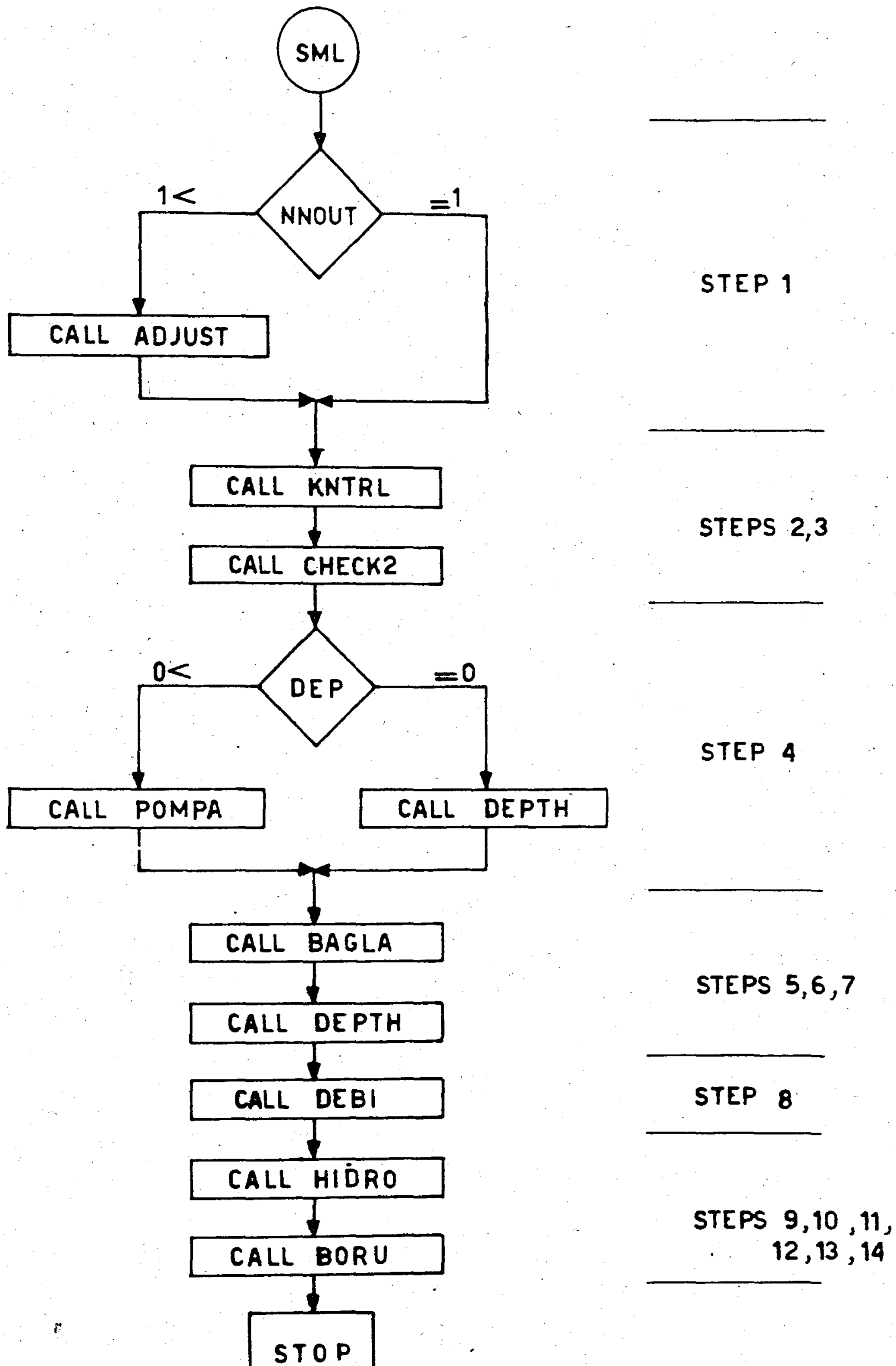


FIGURE 3. Flowchart of the Algorithm for Hydraulic Design of a Specified Main Layout (SML).

below.

STEP 1. Adjustment of data for multi-outlets.

The number of outlets, NNOUT, present in the specified main layout is checked and if a single outlet is declared, meaning that no lift stations will be placed into the system, execution is transferred directly to Step 2.

In case multi-outlets are declared (NNOUT>1), every node, except the last one, in the NOUT array, [NOUT(NNOUT)], will have a lift station. These stations function as local outlets. Then the network data is adjusted by Subroutine ADJUST for each lift station in the following way:

1. A dummy node is generated at the station and assigned a new node number equal to N+1.
2. Elevation of the station node is assigned to the dummy node.
3. Total number of nodes is increased by one.
4. The first node of the draining link is replaced by the number of the dummy node.

It should be noted that the number of links in the main layout, NM, is still equal to the original node number, NO, minus one.

STEP 2. Check data.

Through Subroutine KNTRL, Subroutine CHECK2 is called for each link of the main layout to check for compatibility of the main layout with the spanning tree regulations. If there is no contradiction, execution is transferred to Step 3. Otherwise all the declaration mistakes in the main layout are printed out with appropriate error messages (i.e., "NO EXIT FROM LINK ?"), and then execution is stopped.

STEP 3. Determine initial nodes.

An initial node in a layout is defined as an upper node (in flow direction) of a link to which no link is draining. These nodes serve as the starting points in tracing the routes to be followed when determining nodal depths and discharge rates. The dummy nodes are regarded as initial nodes as well.

Before returning to the main program, Subroutine KNTRL determines the initial nodes by scanning the links of the main layout. It finds all the initial nodes (NIN) and stores the information in an array (array IN) for later use.

STEP 4. Find nodal depths.

If the input variable DEP has been set equal to zero, meaning that no restriction was set for the maximum nodal depths, the algorithm calls Subroutine DEPTH to assign the minimum cover depth, DMIN, to all initial nodes. Beginning with each initial node, all the routes are traced down to an outlet. During the process, the nodal depths are set according to the minimum slope, SMIN, for all links. Nodes resulting with depths smaller than DMIN, are automatically set equal to DMIN.

On the other hand, if a maximum allowable depth has been specified (DEP≠0), the algorithm calls Subroutine POMPA, which begins with each initial node and sets nodal depths according to SMIN upto an outlet, just as Subroutine DEPTH does. Then, in addition to the lift stations specified by the designer, it assigns a lift station to nodes with depths exceeding the allowed limit. After assigning a lift station to a node, Subroutine POMPA updates the network data by increasing the number of outlets:

1. Assigning a dummy node to each lift station and numbering it as N+1.
2. Setting the elevation of the dummy node equal to the elevation of the original node.
3. Increasing the number of nodes by one: N=N+1.
4. Increasing the number of outlets by one: NNOUT = NNOUT+1.
5. Shifting the final outlet in the outlet array (NOUT) so that it still remains as the last one.
6. Placing the original node number of the lift station into the outlet array.

7. Replacing the first node of the draining link by the dummy node.
8. Increasing the number of initial nodes by one: NIN = NIN+1.
9. Storing the dummy node to the initial node array, IN.

The first three and the seventh items are exactly the same adjustments made when multi-outlets are declared. Since assignment of a lift station is equivalent to increasing the number of outlets, these items are repeated here. The other items are necessities for updating the outlet and initial node arrays.

STEP 5. Connect unused links.

The complete network contains M possible links. Only NM (NM = NO-1 or NM = N-NNOUT, where N is the number of nodes) of these links are used in the main layout. Thus, the remaining (M-NM) unused links must be connected to the main layout. This is done by Subroutine BAGLA. In Subroutine BAGLA while connecting each unused link, drainage into an initial node was avoided as much as possible. Otherwise, the depths at the immediate as well as the later nodes would increase until an outlet is reached. If such a case could not be prevented, the connection is made in the direction which requires the least excavation. The total number of initial nodes, NIN, and the inital node array, IN, is updated accordingly.

STEP 6. Find new nodal depths.

After connecting all the unused links to the main layout, Subroutine BAGLA calls Subroutine DEPTH for a second time to retrace all the routes. However, these depths still cannot be considered as final since there is a possibility of changing the slopes during hydraulic computations.

STEP 7. Connect lift stations to downstream sewers.

If a single outlet is specified, the algorithm skips directly to Step 8.

In all the computations so far, the node number of a lift station has been referred to as an outlet and the dummy node assigned there is considered as an initial node for the continuing link. To transfer the flow to the final outlet, the lift station must be connected to the dummy node by a link of zero length. Hence the total number of links, M, is increased by one for each dummy link. Since the dummy links have zero lengths, the already computed depths would remain unchanged.

STEP 8. Determine pipe discharges.

At this step, Subroutine DEBI is called to compute the discharges for each pipe. The horizontal length of each link is multiplied by the flow coefficient (CLOAD), which was read into the program, to find the individual discharges of the corresponding pipes. Then, by a process similar to that used in Subroutine DEPTH, all routes are traced beginning from initial nodes. However, unlike Subroutine DEPTH, where tracing had stopped at any outlet encountered, tracing is continued until the final outlet.

The total discharge for each pipe is computed by summing its own discharge to the contributions from the upstream links.

After completition of this step execution returns back to the main program. Then Subroutine HIDRO is called for the remaining hydraulic computations.

STEP 9. Determine diameter and flow characteristics of each pipe.

At this step of Subroutine HIDRO, Subroutine BORU is called for each pipe to do the iteration explained in Section 3.1.2. The value of Manning's Roughness Coefficient for the pipe material used, required at this step, was read into RN earlier.

To start the iteration process, the pipe diameter should be predetermined. Assuming that the design discharge,

Q_d, is flowing at half depth (Section 3.1.1.4), Subroutine BORU solves Manning's Equation for the pipe diameter. Then this diameter is standardized to the specified minimum sewer size (20 cm) if it is less than 20 cm; for larger sizes computed, the diameter is selected as the next commercially available size exceeding the computed value.

With known pipe diameter and design discharge, angle Θ is computed iteratively. Then, the flow velocity is calculated using the equations given in Figure 2.

<u>STEP 10</u>. Check for maximum and minimum velocities.

Before leaving Subroutine BORU the flow velocity is checked against the velocity limits and:

a) If the maximum velocity is exceeded, the pipe diameter is increased.

b) If the minimum velocity could not be satisfied, the pipe slope is increased by an increment of 0.0005.

In the case of any change in pipe diameter or slope Step 9 is repeated to compute the new hydraulic elements.

Once velocity constraints are satisfied the execution returns back to Subroutine HIDRO with the calculated hydraulic elements of that link.

STEP 11. Determine topographic elements for each link.

Subroutine HIDRO calculates the new nodal (manhole) depth at the lower end of a pipe after a change in slope. Then the excavation volume (V_E) of that link is computed as (Figure 4):

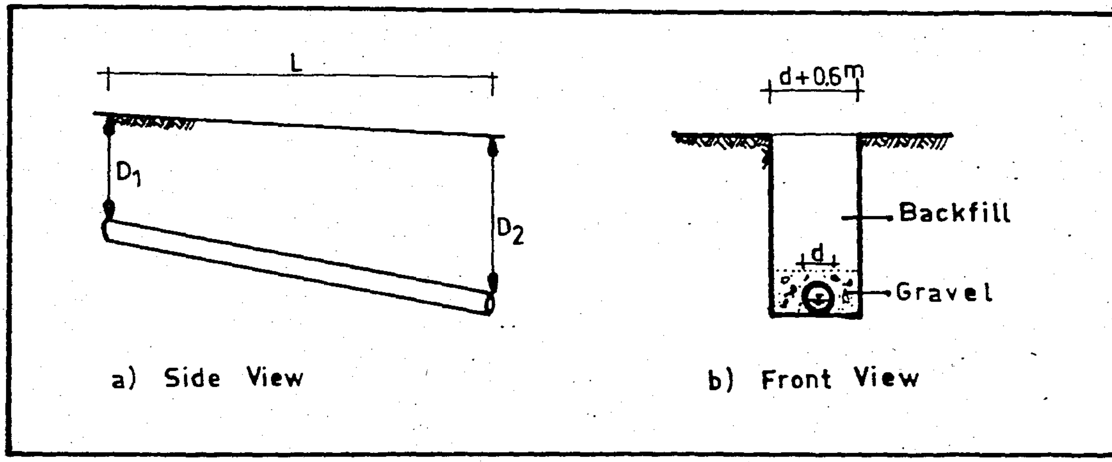


FIGURE 4. Plan and Profile Details of a Sewer.

$$V_E = \frac{D_1 + D_2}{2} * L * (d + 0.6) \qquad (8)$$

where, D_1, D_2 = cover depths of the upper and lower nodes of a link (m); L = horizontal length of a link (m); d = pipe diameter (m). Here 0.30 m of working area is left between each side of the pipe and the trench walls (ASCE, 1969). Finally, when the manhole drop and sewer invert elevations of the handled link are calculated, calculation of all the hydraulic and topographic elements would be complete.

STEP 12. Store information about the full network.

Different layouts can only be compated with parameters which reflect the practicality and the economy of the particular layout. Before handling another link, the following are carried out:

1. Add the sewer length to the previous total if the sewer diameter is equal to the minimum specified size.
2. Add the excavation volume of the link to the previous total.
3. Add the upper nodal depth to the previous total.
4. Compare the upper manhole depth to find the maximum depth in the network and its location.

STEP 13. List the topographic and hydraulic information for each link.

The listing of results, link by link, is aimed to reduce the storage capacity needed for the program. The organization of the listings are consistent with the guidelines of Iller Bankası (1972a).

Execution is transferred to Step 9 until all the links are designed.

STEP 14. Print totals.

After all the links are handled, the totals and the comparison parameters obtained in Step 12 are printed under corresponding headings. Thus the hydraulic design of the network is completed.

3.1.3.2. Design of a Given Full Network

The hydraulic design of a given full network is achieved by Subroutine FULNET. As shown in the flowchart presented in Figure 5, this subroutine is formed by collecting the relevant parts from those subroutines which are not fully needed; hence, it serves as a main program to achieve the execution transfers between existing subroutimes.

The design method is essentially similar to the one presented in Section 3.1.3.1. Several steps, however are excluded due to the following reasons:

i) Since the network layout is specified as a whole data check for the main layout (Step 2) becomes meaningless.

ii) Since flow directions in the network are already specified, the designer needs only the hydraulic design. So, there should not be any unconnected link left (Step 5).

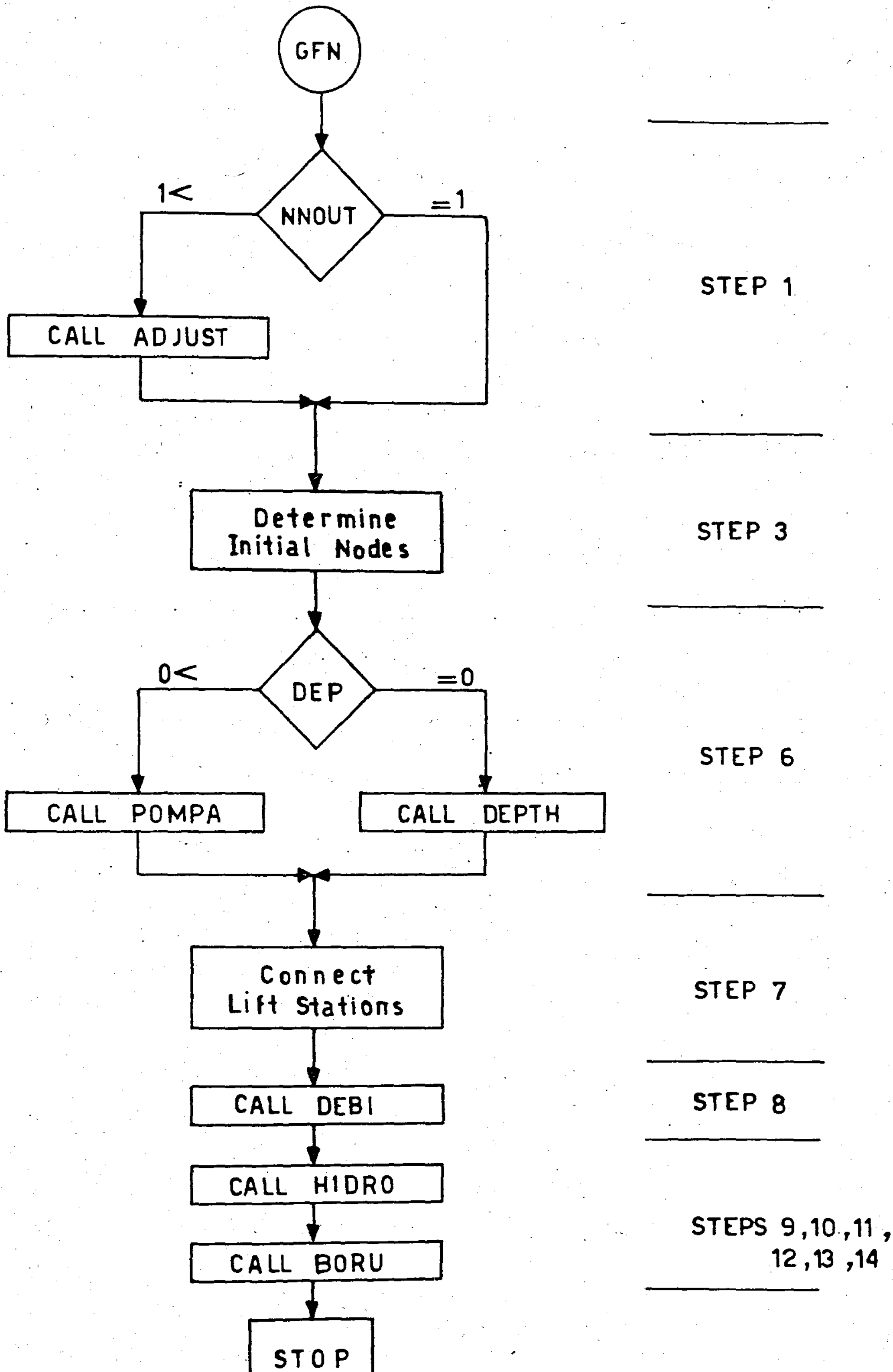


FIGURE 5. Flowchart of the Algorithm for Hydraulic Design of a Given Full Network (GFN) Subroutine FULNET.

iii) Since all links are connected, predetermination of depths (Step 4) needed for completion of missing connections becomes unnecessary.

For convenience the same STEP numbers are retained in the algorithm presented in Sections 3.1.3.1. and 3.1.3.2.

3.2 LAYOUT GENERATION FOR SEWER NETWORKS

Sewers are laid in the direction of the ground slope, with the tributaries lying down the sides of hills towards the main lines which are following the valleys. Shortly, the layout is selected to conform with the topography, with particular emphasis on locating the main lines in valleys. Since the design engineer cannot always be lucky to have a favorable topography, with a valley to drain the area to the outlet, the routing of main lines becomes quite significant in the design process. Once the main collectors are routed, the remaining problem consists of connecting the tributaries to the nearby trunks by considering the topography. Thus, the flow directions, and consequently the excavation costs, depend strictly on the selected main lines.

To reduce the dependence on the designers experience a more consistent procedure, which preferably can be computerized is desired. A literature search for methods to generate main lines for a network turned up the works of Hall and Hammond (1965) and Buras and Schweig (1969). These are summarized here briefly: Hall and Hammond (1965) developed a semi-computerized technique for the optimization of an aqueduct route. An aqueduct in an open channel system is equivalent to a main line of a sewer system. Given a topography, a set of specifications and the unit cost figures, they tried to optimize the route of the aqueduct from a cost stand-point. A dynamic programing algorithm was used to determine the k^{th} best route from all nodes in the aqueduct system to a given node. This analysis yielded a group of feasible routes. Then, these routes were evaluated to select one that has a cost within the desired percentage of the cost of the best route available and satisfies a set of practical constraints not considered when creating the set of feasible routes.

Buras and Schweig (1969) sought an optimal route for the main aqueduct in a water distribution system. They noted that the basic layout of an aqueduct system was like a fishbone: the laterals forming the ribs and the main conduit, the backbone. Without the lateral lines, the routing problem remains as one of aligning the backbone, the main aqueduct to cross the laterals in such a way as to minimize the cost of all components of the system.

In view of the works of Hall and Hammond (1965) and

Lowsley (1973) the idea of generating the main lines by a Shortest Path algorithm occured. Hence the graph theory, from which the shortest paths are developed is reviewed. To develop the optimality criterion required for application of a Shortest Path algorithm, various criteria are derived from characteristics of sewer networks. Then the Layout Generation (LG) algorithm developed using Bellmore's (1972) Shortest Path algorithm, is described in detail.

3.2.1. GRAPH THEORY CONCEPTS

The optimization technique to be presented is based on graph theory. Hence, it will be useful to review the fundamental concepts of graph theory. For detailed information on graph theory the reader is referred to Harary (1969).

A graph may be defined as a collection (or set) of nodes and arcs. Each arc must have a node at either end, but there is no restriction as to the number of arcs connected to any particular node. The two end nodes of an arc are called adjacent nodes.

Every arc has a characteristic length, which is a function of some measurable quantity in the problem at hand. For example, if nodes in a graph represented Izmir and Istanbul, the length of an arc connecting these nodes may represent any one of the following: the distance flown by airplane, the time spent in flying or the flight cost. Likewise, the length of another arc between these two nodes may represent similar criteria for travel by ship. All arc lengths within a graph must be consistent, representing the same measurable quantity.

The arcs may be either directed or undirected. If all the arcs in a graph are undirected, the graph is said to be undirected. Similarly, if all the arcs are directed, the graph is said to be directed. Any graph having both directed and undirected arcs is said to be mixed. Clearly, any given undirected graph can be transformed into a directed graph by replacing each undirected arc with two opposite directed arcs.

A graph is called a planar graph, if it can be drawn on a surface such that none of its arcs intersect except at its nodes. A graph becomes a connected graph, if there is at least one path from each node to every other node. A path is defined as a sequence of arcs between any pair of nodes.

For the remainder of this study, the term graph will mean a "connected, directed, planar graph" unless otherwise noted. Also, if a directed arc joins Node I to Node J, it will be indicated as A_{ij}.

A cycle is a path that begins and ends at the same node. A cyclic graph contains at least one cycle, while an acyclic graph contains none. A connected acyclic graph, i.e., a tree, which contains all nodes of an original graph is called a spanning tree. A spanning tree is called the Minimum Spanning Tree (MST), if the total sum of its arc lengths is minimum. Referring to the graph in Figure 6.a, the MST rooted (having its terminal node) at Node 9 is shown in Figure 6.b.

Path length is defined as the sum of the lengths of the arcs contained in a path. Among the feasible paths tracing the routes from an initial node to a terminal node, the one having the minimum path length is termed as the shortest path. In a similar manner, the shortest path from each node to a root may be found. Applying this procedure to the graph in Figure 6.a, the shortest path between Nodes 1 to 9 is obtained as depicted. The graph shown in Figure 6.c is called the Shortest Path Spanning Tree (SPST) rooted at Node 9.

The following are properties of rooted spanning trees (Harary, 1969):

i) There is one, and only one, path from any given node to the root.

ii) In a tree connecting N nodes, there are N-1 arcs.

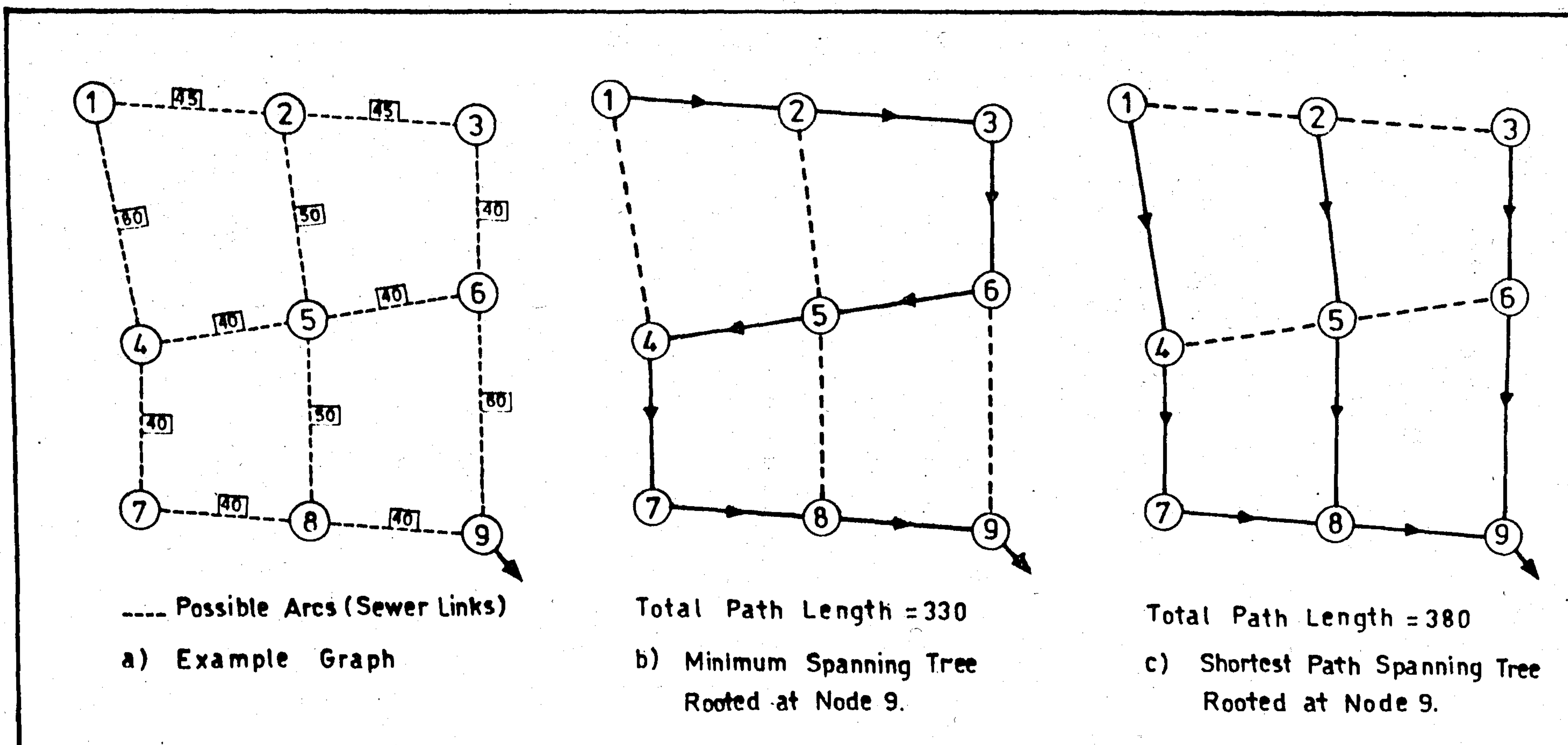


FIGURE 6. The Minimum Spanning Tree and the Shortest Path Spanning Tree of a Graph.

iii) Except the root, each node has one, and only one, arc leaving it.

iv) No arc leaves the root.

In this study, the term "terminal node" is preferred over the term "root" due to its self explanatory meaning in case of a sewer network. Similarly, the term "arc" will be replaced by the term "link".

Using horizontal distance between nodes of planar graph as link length, the MST (Figure 6.b) clearly has the minimum total path length. The SPST (Figure 6.c) is appealing, because its use guarantees minimum distance between any node and a terminal node. In a sewer system SPST thus guarantees the quickest removal of wastewater from any node, and the minimum depth at the terminal node. This is verified in Table 2.

TABLE 2. COMPARISON OF MST AND SPST
FOR A SANITARY SEWER NETWORK

Main Layout	Σ Exc. Vol. (m^3)	Σ Depths (m)	Max Depth (m)	At Node
MST	601	15.91	2.09	9
SPST	543	13.90	1.61	9

Hydraulic design of main layouts derived as MST and SPST for a sanitary sewer system yields the results presented in Table 2. Note that the reduction in total manhole depths has caused a reduction in total excavation volume.

Even though MST results with the minimum path length, it is not appropriate for sanitary sewer networks, since sanitary sewers must be laid in all streets with possible wastewater connections. This approach may be suitable for storm sewers where the runoff may be permitted to flow through gutters in streets with unconnected links.

The problem of finding the shortest path in a network can be solved by using one of the several algorithms developed. An algorithm suggested by Bellmore (1972) is preferred for the problem considered here.

3.2.2. COST ESTIMATES

As noted earlier, determination of a sewer system layout is a problem of optimization. To develop an appropriate optimization criteria, discussion of all relevant cost factors for a sewer systems is in order.

Items affecting total cost of a sewerage system may be grouped as: labor, land acquisition, excavation (rock, earth), manholes, maintenance and repair , materials (pipe, special sections), pump installations, etc. Dajani and Gemmel (1971) pointed out that the cost of supplying and installing the different sewer pipes, the cost of excavation and the cost of manholes constitute the major portion of the construction costs of sewer systems. Baffa (1955) reports that 85% of the cost of gravity flow sewer systems is due to excavation, pipe and installation costs, while the remaining 15% is due to the manholes.

In view of the above discussion, the cost of a gravity flow sewer system can be approximated by:

$$\Sigma \text{ Cost} = \Sigma(\text{Manhole} + \text{Material} + \text{Installation} + \text{Excavation}) \text{ Costs} \qquad (9)$$

Manholes in a sewer network are generally located at regular intervals along sewer links in addition to all nodes. Turkish codes require a maximum interval of 75 m (Iller Bankası, 1972a), whereas American codes permit maximum intervals of 90 to 120 m (ASCE, 1969). In this study, the number and location of manholes are regarded as fixed so that their contribution to the total system cost is constant and thus not subject to optimization.

Pipe costs constitute a significant part of the material costs; however, since pipes are laid in all streets, the total pipe length is fixed for a sewer network. Besides length diameter also affects pipe costs. For relatively small networks the minimum permissible size may be governing. Alper, et al. (1980) reported that a sanitary sewer network with a total length of 41605 m required 40755 m of the minimum permissible size. This corresponds to 98%. Thus increases in size can also be considered insignificant from a cost stand point. Although material costs, due to pipe length and diameter, are beyond the optimization process, the effect of pipe diameter on excavations costs is considered in Equation 8.

Installation costs are a function of soil type, working conditions and sewer depths. Meredith (1972) proposed a series of installation cost equations in terms of unit prices, which are given as functions of average invert depth and pipe diameter. Since the total pipe length and pipe diameter are approximately constant, installation costs will not be dealt with explicitly.

In view of the above discussion the excavation cost is the only remaining item and thus Equation 9 is replaced by:

$$\Sigma \text{ Cost} = \Sigma \, C_i (V_E)_i \qquad (10)$$

where $(V_E)_i$ and C_i are, respectively, the excavation volume and unit excavation cost for each link. For uniform soil type and working conditions the unit excavation cost assumes a constant value and reduces to:

$$\Sigma \text{ Cost} = C \; \Sigma \; (V_E)_i \tag{11}$$

Furthermore, assuming a uniform slope between adjacent nodes, the excavation volume of each link can be written as in Equation 8. Figure 7 shows that Equation 8 is valid for both type of surfaces.

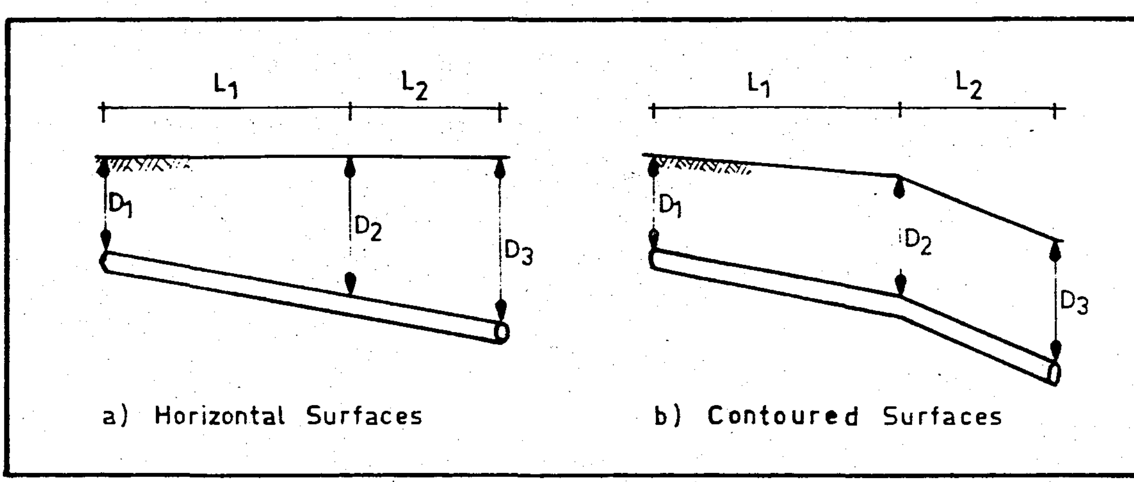


FIGURE 7. Plan of Sewer Locations for Horizontal and Contoured Surfaces.

Combining Equation 8 with Equation 11 for a network with M links yields the following:

$$\Sigma \text{ Cost} = C \cdot \sum_{i=1}^{M} \frac{(D_1 + D_2)_i}{2} \cdot L_i \cdot (d + 0.6)_i \tag{12}$$

Since the effect of diameter variations on cost is insignificant Equation 12 may be reduced to:

$$\Sigma \text{ Cost} = \frac{1}{2} C(d + 0.6) \sum_{i=1}^{M} (D_1 + D_2)_i \, L_i \qquad (13)$$

Since the objective is cost minimization, the constants may be eliminated and the objective function may be approximated as:

$$\text{Min } \Sigma \text{ Cost} \sim \text{Min} \sum_{i=1}^{M} (D_1 + D_2)_i \, L_i \qquad (14)$$

Unfortunately, the objective function expressed in Equation 14 is pretty difficult to deal with. To illustrate this point consider the simple case of uniform horizontal lengths (i.e., $L_i = L$ for all i). Then Equation 14 reduces to:

$$\text{Min } \Sigma \text{ Cost} \sim \text{Min} \sum_{i=1}^{M} (D_1 + D_2)_i \qquad (15)$$

or equivalently:

$$\text{Min } \Sigma \text{ Cost} \sim \text{Min} \sum_{i=1}^{N} D_i \qquad (16)$$

Equation 16 states that the layout with the minimum total nodal depths will approximate the layout with the minimum cost. However, the nodal depths are unknown prior to the selection of the layout. Therefore, it is impossible to solve the problem within this framework.

Several attempts have been made to get around this difficulty. For example, Joneja et al. (1978) developed a cost function depending on trench depth and pipe diameter which obviously cannot be used unless the layout is specified. Dong (1980) presented an optimization technique, which varies both the pipe diameter and slope to obtain a least-cost combination. However, although such a combination may be found for individual pipes; for pipes in series, an unnecessary increase in the slope of an upstream pipe will increase the excavation costs of all the downstream pipes. In short, the actual excavation cost of eack link cannot be assigned due to unknown invert elevations. Thus the following question arises: "What must be assigned as a measure of the cost of each link?". Hence, an appropriate cost must be developed from the known data, even if it is only an indirect measure of actual excavation costs.

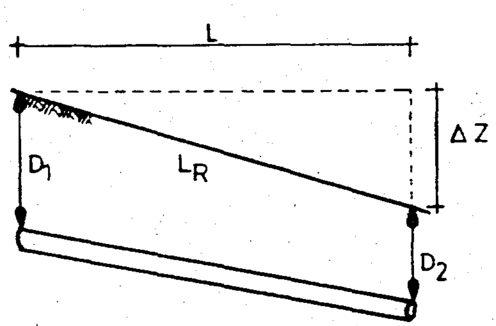
In sewer systems, the purpose is to drain the sewage to the final outlet as quickly as possible and thus along the shortest paths. Since flow travel time depends on the slope of the sewers and the path lengths, the total path length is the parameter characterizing a particular layout. The only known data, prior to hydraulic design, consists of the horizontal lengths of the sewers and the surface elevations of the nodes. The criteria which can be developed from this data includes the following:

1) Assigning the horizontal length of each link as its cost.

In this case the algorithm will drain each node to the outlet through the path having the shortest length. For a flat drainage area, this method guarantees the minimum depth at the outlet. However, over contoured surfaces the resulting main layout may be far from optimum, due to exclusion of the natural slope. This criterion will be referred to as HL.

2) Assigning the real surface length of each link as its cost.

To include the effect of natural topography, assingment of real surace lengths may be more realistic. These lengths may be computed as illustrated below:



$$L_R = \sqrt{L^2 + \Delta Z^2}$$

FIGURE 8. Explanation of Real Surface Lengths.

This run will be referred to as RL.

3) Assigning the inverse of the natural slope as the link cost.

Laying the main lines in the direction of maximum surface slope can reduce the necessary excavation volume as illustrated in Figure 9. If the natural slope (SN) between two adjacent nodes is equal to SMIN, the pipe slope will also be equal to SMIN (Figure 9.a). (The increase of pipe slope due to VMIN requirement is not considered.) However, if the natural slope is greater than SMIN, the pipe slope may be equal to or less than SN (Figure 9.b). Both cases reduce the invert elevation of the lower node of a pipe again reducing the excavation volume. Since Shortest Path is a minimization algorithm, this may be achieved by assigning the inverse of the natural slope as the link cost:

$$\begin{aligned} IS &= 1/SN \\ &= 1/(\Delta Z/L) \\ &= L/\Delta Z \qquad (17) \end{aligned}$$

For links having an elevation difference more than 1.0 m between its nodes, the cost in Equation 17 is reduced. To prevent large increases in cost for smaller elevation differences, horizontal lengths are assigned as the corresponding costs in both directions (i.e., ΔZ is

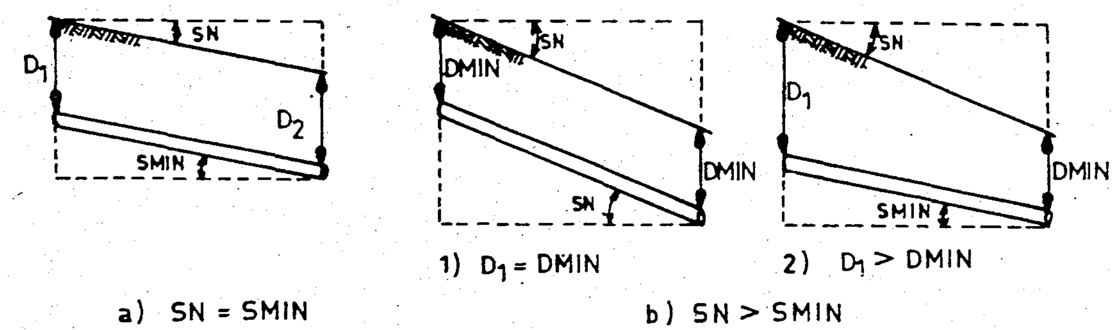


FIGURE 9. Changing of Pipe Slope with Natural Slope.

set to unity).

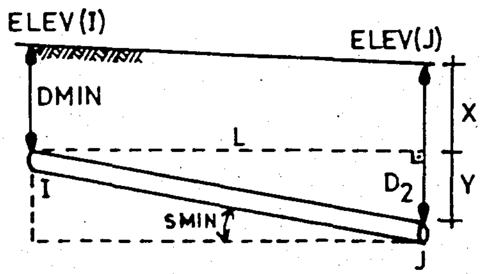
This run will be referred to as IS.

4) Assigning a hypothetical excavation cost for each link.

A hypothetical excavation cost can be assigned to each link by setting the depth of the upstream node to DMIN and laying the sewer at SMIN. By doing so, the downstream depth is calculated and the excavation volume (an excavation cost) can be computed in the following way as shown in Figure 10.

This excavation cost is true only for the sewers draining the initial nodes at minimum slope. This run will be referred to as EX.

In all four criteria developed above, the flow direc-



$$Y = SMIN \times L$$
$$X = ELEV(J) - (ELEV(I) - DMIN)$$
$$D_2 = X + Y$$
$$\text{IF } D_2 < DMIN \Rightarrow D_2 = DMIN$$
$$EX = \frac{DMIN + D_2}{2} \times L$$

FIGURE 10. Computation of Excavation Cost.

tion is always set in the direction of ground slope. If the flow in any sewer is not consistent with the natural slope it will be called as adverse flow. Adverse flow is permitted only in links with elevation differences less than 1.0 m between its nodes. The same cost is assigned in the adverse flow direction as well. This is the true case for the first two criteria. It is self adjusted in the third criterion, but it does not reflect the true case for the fourth criterion. Thus EX criterion is modified to assign the corresponding excavation cost when adverse flow is permitted. This cost is calculated using again the method explained in Figure 10.

The above criteria will be applied to networks of various sizes available in the literature. The results obtained will be compared with one-another and with the solutions presented for these networks. The comparisons will be assessed to select the best criterion for generating layouts with the least excavation volume.

3.2.3. AN ALGORITHM FOR LAYOUT GENERATION

The generated layout for a network is to be made up of the Shortest Path Spanning Tree of that network. Links join the nodes with a cost penalty and they may be either directed or undirected.

Due to the high storage capacity needed, the basic algorithm is not suitable for large networks. Thus the layout generation (LG) algorithm has been modified to analyze both small and large networks separately. The flowchart of the LG algorithm is presented in Figure 11.

3.2.3.1. For Small Networks

Layout generation for small networks is achieved using an algorithm made of five subroutines: INILAY, CHECK1, PATHS, DECODE and LOOP. The major steps of the algorithm are listed and described below. As a further aid in understanding the structure of the algorithm, an example network is worked out in conjunction with the steps of the algorithm. The generation of the layout as well as the design of this example is presented in the Appendix to illustrate the data preparation and to show the output obtained from the program.

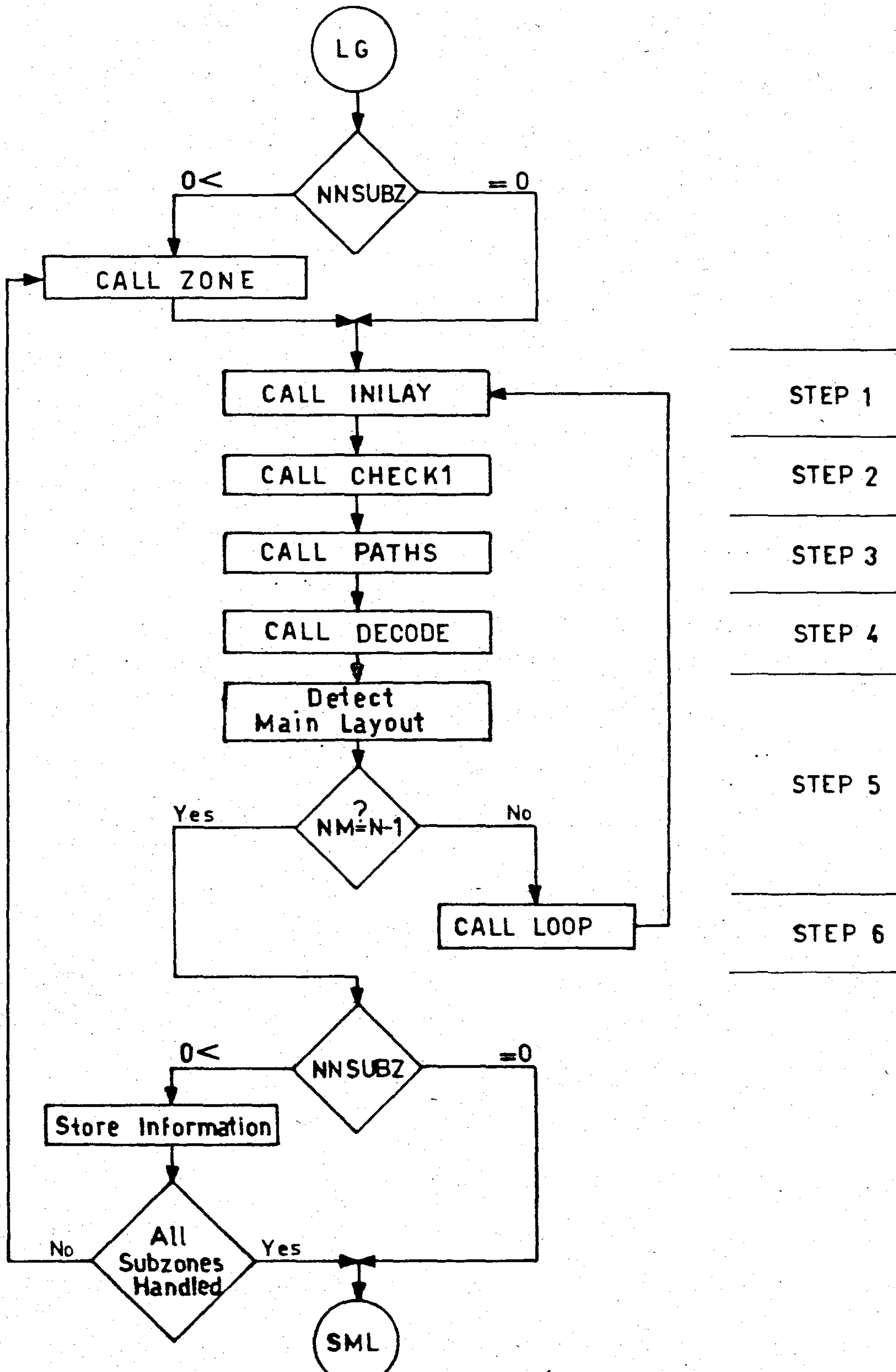


FIGURE 11. Flowchart of the Layout Generation (LG) Algorithm (for main layout).

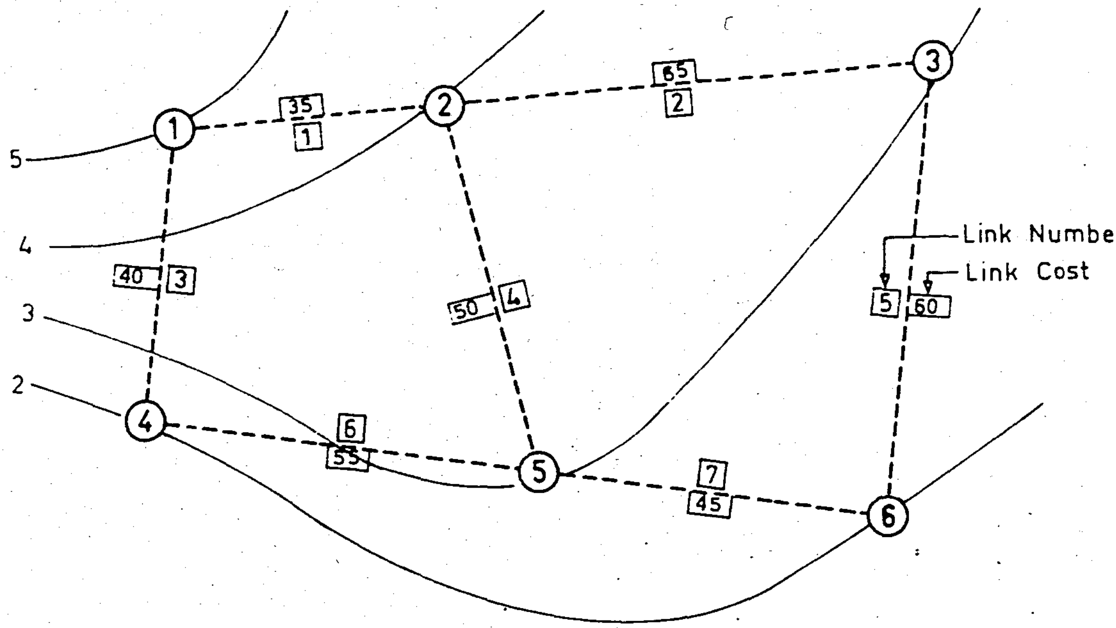
STEP 1. Assign flow directions.

Flow direction in each sewer is assigned in accordance with the surface elevations. Initially, flow is permitted in the direction of natural ground slope. As a weight factor, or cost, one of the criteria discussed in Section 3.2.2 is assigned to each link to generate the weighted adjaceny matrix, DIST, of the network. This matrix has dimensions of (NxN) and later, it will be used to store the information on the shortest paths.

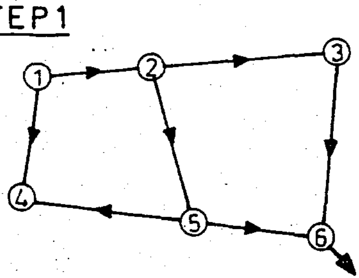
This step is completed by the initilization of two matrices IPATH(NxN) and ICONT(Mx1) which are to be used in the upcoming steps. IPATH is initialized by having every row set to 1,2,3,...,N, whereas ICONT is initialized by setting every entry to unity.

STEP 2. Check all nodes for a drainage link.

Due to the assignment of flow directions in accordance with the topography, some nodes (i.e., nodes whose elevation is less than the elevations of the adjacent nodes) may not have a drainage link and may act as a sink to that section of the network. Node 4 in Figure 12.a shows such a case. Subroutine CHECK1 detects such sink-nodes and assigns adverse flows to all links leading from these nodes. Figure 12.e represents the resulting network.



STEP 1



a) NETWORK

$$
\begin{array}{c|c}
 & \text{To Node} \\
\text{From Node} & \begin{array}{cccccc} 1 & 2 & 3 & 4 & 5 & 6 \end{array} \\
\begin{array}{c} 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \end{array} &
\begin{bmatrix}
\infty & 35 & \infty & 40 & \infty & \infty \\
\infty & \infty & 65 & \infty & 50 & \infty \\
\infty & \infty & \infty & \infty & 60 & \infty \\
\infty & \infty & \infty & \infty & \infty & \infty \\
\infty & \infty & \infty & 55 & \infty & 45 \\
\infty & \infty & \infty & \infty & \infty & \infty
\end{bmatrix}
\end{array}
$$

b) DIST Matrix

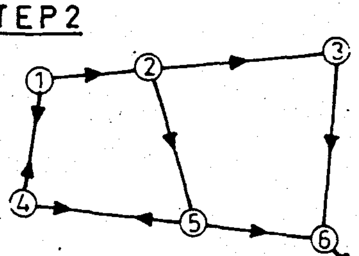
$$
\begin{array}{c|c}
 & \text{To Node} \\
\text{From Node} & \begin{array}{cccccc} 1 & 2 & 3 & 4 & 5 & 6 \end{array} \\
\begin{array}{c} 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \end{array} &
\begin{bmatrix}
1 & 2 & 3 & 4 & 5 & 6 \\
1 & 2 & 3 & 4 & 5 & 6 \\
1 & 2 & 3 & 4 & 5 & 6 \\
1 & 2 & 3 & 4 & 5 & 6 \\
1 & 2 & 3 & 4 & 5 & 6 \\
1 & 2 & 3 & 4 & 5 & 6
\end{bmatrix}
\end{array}
$$

c) IPATH Matrix

$$
\begin{array}{c c}
\begin{array}{c} 1 \\ \\ \\ \\ M \end{array} &
\begin{bmatrix}
1 \\ \vdots \\ \vdots \\ \vdots \\ 1
\end{bmatrix}
\end{array}
$$

d) ICONT Matrix

STEP 2



e) NETWORK

$$
\begin{bmatrix}
\infty & 35 & \infty & 40 & \infty & \infty \\
\infty & \infty & 65 & \infty & 50 & \infty \\
\infty & \infty & \infty & \infty & 60 & \infty \\
40 & \infty & \infty & \infty & 55 & \infty \\
\infty & \infty & \infty & 55 & \infty & 45 \\
\infty & \infty & \infty & \infty & \infty & \infty
\end{bmatrix}
$$

f) DIST Matrix

$\infty = 99{,}999{,}999.$

FIGURE 12. Example Problem.

STEP3

$$\begin{bmatrix} \infty & 35 & 100 & 40 & 85 & \infty \\ \infty & \infty & 65 & \infty & 50 & \infty \\ \infty & \infty & \infty & \infty & \infty & 60 \\ 40 & 75 & \infty & \infty & 55 & \infty \\ \infty & \infty & \infty & 55 & \infty & 45 \\ \infty & \infty & \infty & \infty & \infty & \infty \end{bmatrix} \qquad \begin{bmatrix} 1 & 2 & 2 & 4 & 2 & 6 \\ 1 & 2 & 3 & 4 & 5 & 6 \\ 1 & 2 & 3 & 4 & 5 & 6 \\ 1 & 1 & 3 & 4 & 5 & 6 \\ 1 & 2 & 3 & 4 & 5 & 6 \\ 1 & 2 & 3 & 4 & 5 & 6 \end{bmatrix}$$

g) DIST Matrix   h) IPATH Matrix

$$\begin{bmatrix} \infty & 35 & 100 & 40 & 85 & 130 \\ 145 & \infty & 65 & 105 & 50 & 95 \\ \infty & \infty & \infty & \infty & \infty & 60 \\ 40 & 75 & 140 & \infty & 55 & 100 \\ 95 & 130 & 195 & 55 & \infty & 45 \\ \infty & \infty & \infty & \infty & \infty & \infty \end{bmatrix} \qquad \begin{bmatrix} 1 & 2 & 2 & 4 & 2 & 2 \\ 5 & 2 & 3 & 5 & 5 & 5 \\ 1 & 2 & 3 & 4 & 5 & 6 \\ 1 & 1 & 1 & 4 & 5 & 5 \\ 4 & 4 & 4 & 4 & 5 & 6 \\ 1 & 2 & 3 & 4 & 5 & 6 \end{bmatrix}$$

i) DIST Matrix   j) IPATH Matrix

STEP4

From Node 1 To Node 6

$$\begin{bmatrix} 1 \\ 2 \\ 5 \\ 6 \end{bmatrix}$$

	Storage	Flow Direction
Link 1 →	ICONT(1) = 0	NODE (1,1) = 1 NODE (1,2) = 2
Link 4 →	ICONT(4) = 0	NODE (4,1) = 2 NODE (4,2) = 5
Link 7 →	ICONT(7) = 0	NODE (7,1) = 5 NODE (7,2) = 6

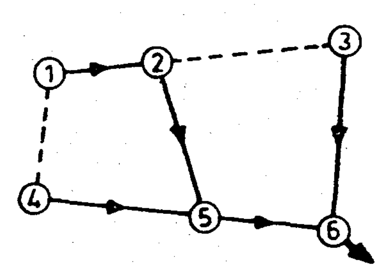
k) JPATH Matrix   l) ICONT Matrix   m) NODE Matrix

From Node 5 To Node 6 $\begin{bmatrix} 5 \\ 6 \end{bmatrix}$ Link 7 → ✓   —

n) JPATH Matrix

STEP5

Count ICONT(1) = 0 $\begin{bmatrix} 1 \\ 4 \\ 5 \\ 6 \\ 7 \end{bmatrix}$ $\left.\begin{matrix} NM = 5 \\ N-1 = 6-1 = 5 \end{matrix}\right\}$ ✓ ⇒



p) LAYOUT Matrix   r) Main Layout

FIGURE 12. Continued.

Another problem that may occur is the existence of drainage links at the outlet. This type of a situation may arise when the elevation of the outlet is higher than the elevation of the adjacent nodes. Subroutine CHECK1 again changes the flow directions of all links to drain them to the outlet.

STEP 3. Find the shortest paths.

Subroutine PATHS is called to find the shortest paths between every pair of nodes in the network. The output of Subroutine PATHS consits of the final form of the DIST and the IPATH matrices. The DIST matrix provides information on total length, or cost, of a path, whereas the IPATH matrix is used in tracing the shortest paths. A matrix entry DIST(I,J) gives the total length, or cost, of the shorest path from Node I to Node J. On the other hand, the matrix entry IPATH(I,J) gives the first node visited along the shortest path from Node I to Node J. Figure 12.i and 12.j give the final DIST and IPATH matrices for the example network. For example, the shortest path from Node 1 to 6 has a total length of 130 units (=DIST(1,6)). The node visited first along the shortest path is Node 2 (=IPATH(1,6)), followed by Node 5 (=IPATH(2,6)) and by Node 6 (=IPATH(5,6)). Thus, the shortest path is specified as 1-2-5-6.

Incidentally, the initialized state of IPATH

(Figure 12.c) assumes that every node is joined to every other node through a single link. Clearly, if this is not possible the cost of going over such a link is taken as a very large number (Figure 12.b).

STEP 4. Decode path information.

The shortest path from each node to the specified outlet node is decoded by Subroutine DECODE. In decoding each path all the visited nodes including the initial and terminal ones, are stored into vector JPATH. For example, the shortest path from Node 1 to Node 6 is stored as Figure 12.k. From this information the links traced on the way (crossed links) are identified and recorded into ICONT vector. The recording of traced links is done by setting ICONT(M)=0 for link M. Before decoding a new path, the final flow directions of the related links are corrected in NODE array in view of the shortest path (Figure 12.m).

STEP 5. Map the links marked in ICONT to generate SPST.

All the marked links in ICONT are counted to see if there is exactly N-1 links (Spanning Tree Specifications 2). If so, these N-1 links are stored into the LAYOUT array as the generated layout. This completes the layout generation part and execution is transferred to the main program.

If there is less than N-1 links, there must be a group of out-of-tree nodes, which form a closed loop and cannot drain to the outlet. Since the flow directions are assigned in accordance with the ground slope (Step1) and Node 53 is locally lower than the adjacent nodes (Figure 13.a), Node 53 cannot drain. This was checked in Step2 and adverse flows were allowed in all the links connected (links 67 and 68). As seen in Figure 13.b, in spite of the adverse flows allowed these links have no connection to the outlet, forming a closed loop.

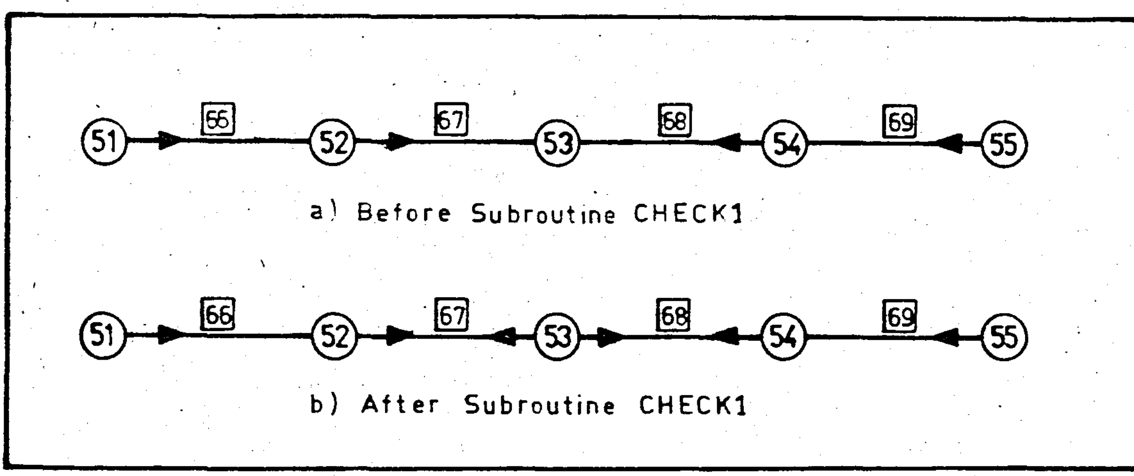


FIGURE 13. Formation of Out-of-Tree Nodes (Closed Loop) in a Network.

<u>STEP 6</u>. Identify closed loops.

Subroutine LOOP is called at this step to detect the closed loops present in the network. Such nodes are identified and execution is returned to Step1 for a rerun. During the rerun process, at Step2, Subroutine CHECK permits adverse flows in those links leading to the nodes of the closed loop. When Step5 is reached, the number of links equals N-1.

3.2.3.2. For Large Networks

Due to limited storage capacity, layout generation for large networks may not be possible with the existing program. The storage could be increased by increasing the matrix size; however, this would lead to excessive storage capacity demands. This problem can be avoided by subzoning the network.

The number of subzones created is specified in variable NNSUBZ. Then each subzone is defined by specifying the nodes contained. Another way of dividing the network into subzones, may be by separating links into groups.

Declaration of zero for NNSUBZ means that the network size is within the capacity of the program and this will be solved as a whole. Then the network information, read into the working arrays, is sent directly to Subroutine INILAY to generate the layout of the network.

In case subzoning is required, information of each subzone must be sent to Sburoutine INILAY in working arrays. Thus, to store the original input, the complete network information is copied into suitable arrays if subzoning is required.

Subroutine ZONE is called from the main program, to read and prepare the first subzone for layout generation. This is achieved by taking out the necessary values from storage and renumbering them consecutively, since all existing subprograms work with ordered link and node numbers.

Another problem that may arise is the double counting of a link by including it into more than one subzone. To prevent this the flow direction in each link is set considering the topography and the upper and lower nodes of that link are stored into NODEF array, accordingly. No change is allowed in this array afterwards.

Subroutine ZONE follows the sequence below:

1) Read the node numbers of a subzone into array NF.

2) Read local outlet, NUH.

3) Obtain the link numbers from array NODEF: If the second node of any link is among the declared nodes of the subzone then the link number is stored into MF(M) array. Here M is the total number of links in a subzone. It is initialized to zero at the beginning and incremented by one as each link is checked.

4) Existence of a link in a subzone requires both the initial and final nodes of that link to be in that sub-

zone. The first nodes of the links stored in the MF array is also checked to see if they are among the nodes of the subzone. The missing nodes are added to the existing nodes of the subzone. While searching for the links, the links that are drained to such nodes need not be considered as a member of that subzone. This is necessary to prevent the inclusion of nodes which are not adjacent to the declared nodes of the handled subzone.

5) The new numbers assigned for each node and link represent their order. For example, NF(5)=17 means that Node 5 was previously Node 17.

6) Find the corresponding length and elevation for each link from arrays HLC and ELEVC, both of which contain the original data.

7) Fill the NODE array with the new node numbers.

After these steps, the data for this subzone is ready for layout generation. The procedure described in Section 3.2.3.1 can now be applied. The flow directions determined for this subzone are stored in NODET array. The link numbers contained in this generated layout are stored in LAYO array. Both sets of information are saved in terms of the original node and link numbers.

The interconnections among subzones is achieved by declaring the outlet of one subzone (say Subzone 2) among the nodes of the neighboring subzone (say Subzone 1). The outlet node of Subzone 2, along with all the other nodes in Subzone 1 will be drained into the outlet of Subzone 1.

After the layouts of all subzones are generated and the results placed into the final collection arrays, this data is transferred back to the work arrays (NODE, LAYOUT, ELEV and HL) to be used in the hydraulic design algorithm.

4. APPLICATION OF THE PROGRAM AND COMPUTATIONAL RESULTS

To implement the criteria discussed in Section 3.2, a program has been written in FORTRAN IV. This computer program is presently dimensioned to handle a network with up to 600 nodes and 600 links. However, for layout generation, such a network must be divided into subzones, each having a maximum of 70 nodes.

In this chapter, first the optimality criterion selected is tested by applying it to networks with various sizes. Then, for proper application of the layout generation algorithm, guidelines are developed and presented for data preparation.

4.1. SELECTION OF OPTIMALITY CRITERION

Based on the various applications of the test criteria the one yielding the minimum excavation volume will be selected as the optimality criterion.

The selected criteria are first tested with Liebman's (1967) trial network (Figure 14). The data is converted

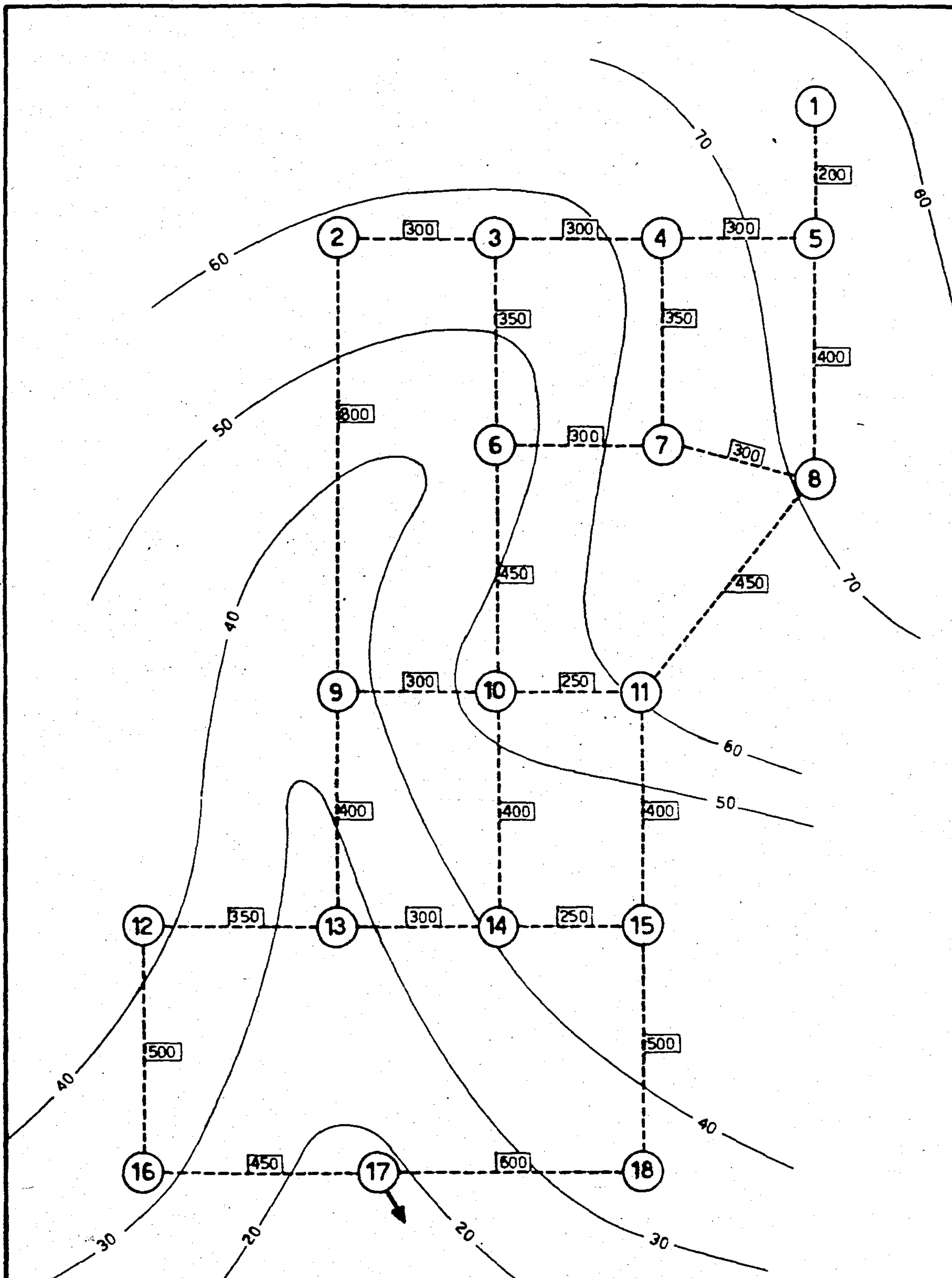


FIGURE 14. Topography of Liebman's Network. (Dimensions are in ft).

into SI units. To compare the results obtained, Liebman's trial layout (dark lines in Figure 15) is taken as the given main layout, and designed. Comparison of these runs is presented in Table 3.

TABLE 3. COMPARISON OF LIEBMAN'S NETWORK WITH THE CRITERIA ADAPTED FOR LAYOUT GENERATION

Data	Criteria	Σ Exc. Vol. (m^3)	Σ Depths (m)	Max Depth(m)	At Node
Liebman's Network	GIVEN LAY.	3152	32.88	5.16	12
	RL	3285	33.64	5.16	12
	HL	3254	33.64	5.16	12
	IS	3183	32.88	5.16	12
	EX	3152	32.88	5.16	12

Results are compared with respect to the following criteria: total excavation volume, total manhole depths and the maximum manhole depth. Among these, the total excavation volume is the most significant one due to its domination of the system cost. Although, all four criteria show similar variations, total excavation volume has a wider range to reflect the small differences among the criteria tested. This can be seen between HL and RL criteria, and IS and EX criteria in Table 4. Table 3 shows that RL, HL and IS criteria yield poor layouts in

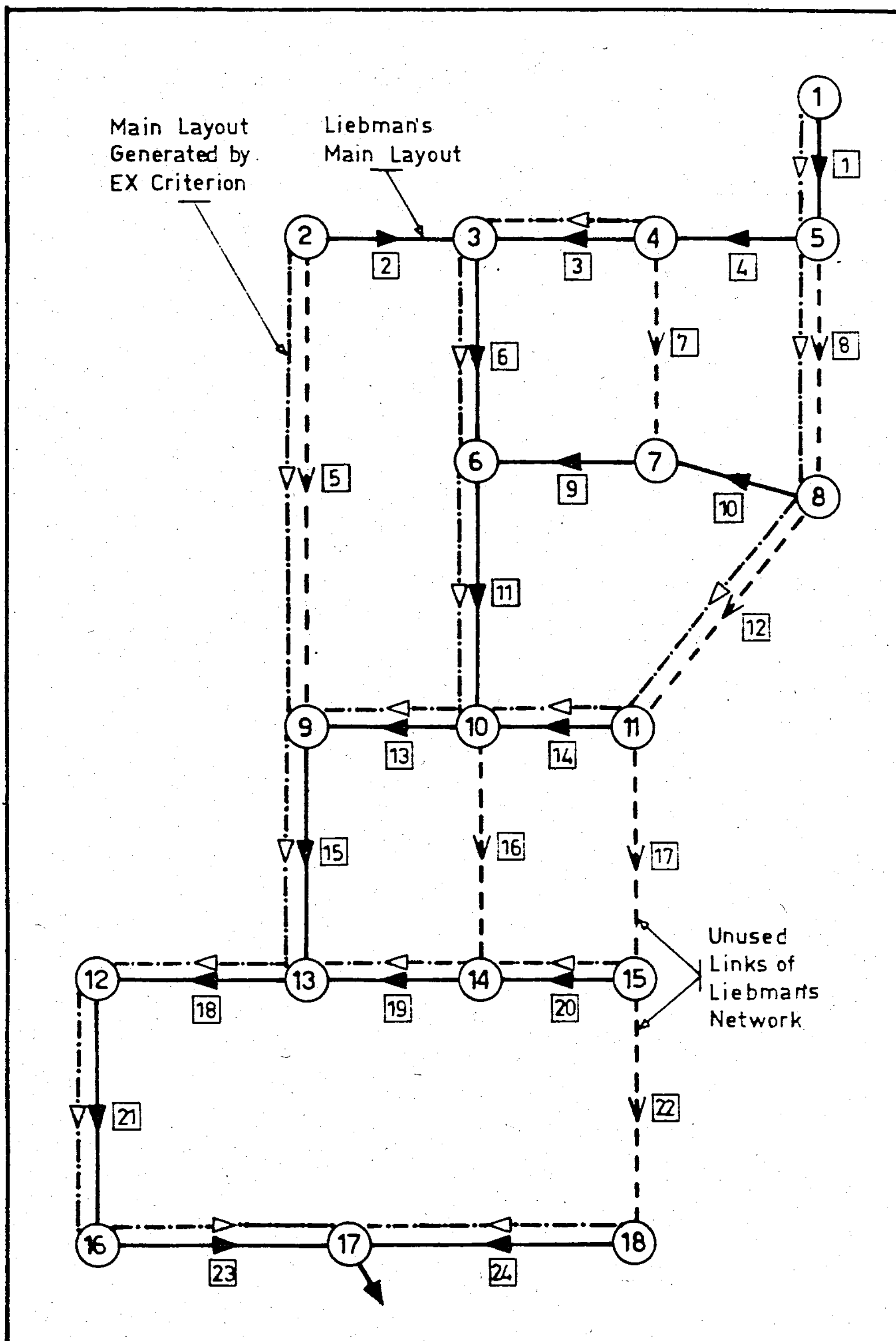


FIGURE 15. Main Layouts Generated by Liebman (1967) and EX Criterion.

comparison with Liebman's; however, EX criterion matched Liebman's layout. The main layout, generated by EX criterion and shown in Figure 15, is different that Liberman's layout.

Liebman (1967) stated that the computer execution time for trials ranged from 30 sec. to 90 sec., depending on the number of exchanges required. Execution time is directly affected when started with a poor initial layout. Here, the execution time for the run with the EX criteria was 1.97 sec on a UNIVAC 1106. Since the layout generation process is the same for all criteria, the execution time does not change among them.

Performance of the existing criteria was then tested on medium sized networks. Here the term is used to refer to networks with 70 or less nodes. For these networks, layout can be generated without subzoning. Three different medium sized networks with preselected main layouts were designed by three different students. For these networks, the optimal layouts were generated and the networks designed to compare with the results of their given layouts (Table 4).

Table 4 shows that the RL, HL and IS criteria yield similar results for medium sized networks. The EX criterion yielded the best layouts for all cases. One may question

TABLE 4. EFFECT OF OPTIMALITY CRITERIA FOR THREE MEDIUM SIZED NETWORKS

Network	Criteria	Σ Exc. Vol. (m^3)	Σ Depths (m)	Max Depth(m)	At Node
M1 (N = 48, M = 68)	GIVEN LAY	5632	80.70	3.38	36
	RL,HL,IS	5632	80.70	3 38	36
	EX	5553	78.35	3 95	36
M2 (N = 40, M = 59)	GIVEN LAY	6658	87.36	3.93	39
	RL HL	6197	86.12	3.03	39
	EX, IS	5986	81.20	3.06	39
M3 (N = 41, M = 58)	GIVEN LAY	7315	94 47	4,77	36
	RL, HL, IS	65 86	89.48	3.53	22
	EX	6179	86.31	3.53	22

the acceptability of the given layouts. The given layouts were checked and found to be realistic and suitable.

The EX criterion shows a 1136 m^2 decrease in excavation volume in the largest network, M3. This corresponds to a 15.5% improvement in the given layout.

The execution time varied in the range of 7-13 secs, changing with the size of the network. This time includes both the layout generation and hydraulic design process.

However, the execution time for only the hydraulic design process (which corresponds to given layouts) ranged from 3 to 5 seconds.

Finally, the existing criteria are applied to the sanitary sewer network of a 13,000 resort town, Pınarkent, presented by Alper et al. (1980). This network was solved and a layout was recommended by the designers. This layout is specified as the given layout. However, since Pınarkent had 312 nodes, layout generation was possible only by subzoning. This network was divided into 8 subzones and a set of runs were obtained. Results are presented in Table 5. For these runs, the outlet was at Node 200.

TABLE 5. RESULTS OF THE TEST CRITERIA FOR PINARKENT

Data	Criteria	Σ Exc. Vol. (m^3)	Σ Depths (m)	Max Depth(m)	At Node
PINARKENT	GIVEN LAY	53 240	822.70	8.56	127
PINARKENT (8 Subzones)	RL	57 674	868.85	8.27	127
	HL	57 302	868.96	8.27	127
	IS	55 948	850.20	8.27	127
	EX	55 141	839.20	8.34	127

Although the EX criterion consistently yielded better layouts in comparison with the other's, it is still inferior to the given layout. This results was somewhat expected and may be explained as floows: the LG algorithm searched for the best layout in each subzone. The layout obtained by combining the generated layout of each subzone is not necessarily the best one for the whole layout. The main reason for this result may be the lack of enough interconnections among the subzones. Draining each subzone by a single outlet may have induced large depths in the adjacent subzone draining this outlet.

As a result of the applications presented above, use of the EX criterion can be recommended. However, it must be emphasized that this criterion does not necessarily yield the optimum layout since it relies on a hypothetical excavation volume, as previously discussed in Section 3.2.2. From here on, the influence of all the other parameters will be investigated using only the EX criterion.

4.2. GUIDELINES FOR THE PREPARATION OF DATA

One of the aims of this study is to reduce the role of engineering judgement in sewer system design. Thus the layout generation algorithm has been developed. As a major advantage over the previous work (Liebman, 1967;

Lowsley, 1973), the present algorithm does not require an initial layout. Specifying the drainage area topography, is sufficient for the algorithm developed here. To simplify the application of this algorithm some control parameters are specified. These parameters will be discussed in this section. Also, to improve the layout generated, a few guidelines in data preparation will be presented.

4.2.1. SITING AND NUMBER OF LIFT STATIONS

Specification of multi-outlets for a given layout is a routine task. The lift stations are predetermined and the layout is specified accordingly. While declaring multi-outlets for a generated layout, special care should be taken. Specification of a node for siting a lift station without considering its position in the generated layout may reduce the expected benefits from that lift station. To find the appropriate sites for lift stations a layout may be generated and designed with only the final outlet specified. This layout may be studied to select the sites for lift stations. Then the layout must be redesigned to reflect the effect of the lift station on the sewer lines.

In Table 6, the given layout is created in accordance with a predetermined lift station at Node 119. Specification of this node as an outlet on the generated layout resulted in 940 m^3 of additional excavation in comparison

TABLE 6. EFFECT OF OUTLET LOCATIONS

Data	Nodes of Lift Stations	Criteria	Σ Exc. Vol. (m^3)	Σ Depths (m)	Max Depth(m)	At Node
PINARKENT	119	GIVEN LAY	50 295	796.86	6.23	80
PINARKENT	119	EX	53 933	809.60	6.52	118
(8 Subzones)	115	EX	52 993	792.68	5.82	106

with the declaration of Node 115 instead of Node 119. Node 115 is the common outlet of two adjacent subzones. This choice led to a 1.74% decrease in the total excavation.

The existence of an additional outlet (or a lift station) in a network always decreases the total excavation volume. Hence, although the excavation costs are reduced the total cost may increase due to the cost of the pump station placed. In such cases, the designer should consider the overall economy. Unless the additional cost of a lift station is worth the benefits obtained, it should be avoided. Alper et al. (1980) made such an analysis and found that two pumps for this network is the most suitable one.

4.2.2. DEPTH CRITERIA

To investigate the effect of DEP parameter a set of runs are obtained for PINARKENT. This parameter is tested on the layout generated in 8 subzones with EX criterion. Results are presented in Table 7.

TABLE 7. EFFECT OF MAXIMUM ALLOWED DEPTH (DEP)

Data	DEP (m)	Number of Lift Stations	Σ Exc. Vol. (m^3)	Σ Depths (m)	Max Depth (m)	At Node
PINARKENT (8 Subzones)	4.0	7	49 858	735.08	4.13	50
	4.5	4	51 424	766.44	4.48	119
	5.0	2	52 711	783.69	4.94	121
	5.5	2	52 389	778.13	5.37	123
	6.0	1	53 176	794.65	5.91	117

The increase of the excavation volume with the increase of maximum allowed depth is expected due to the decrease in the number of lift stations. For DEP=4.0 and 4.5 m it can be seen that the decrease achieved in total excavation volume may not be worth the building of 7 or 4 lift stations. This decision is a matter of economic analysis. In case of DEP=5.0 and 5.5 m the total excavation volume decreased in spite of the increase in the allowed maximum depth. This decrease is due to better siting of the same number of lift stations (2 here). The last DEP restriction can be satisfied with a single lift station.

Along with automatic siting, simultaneous siting of prescribed lift stations is considered. Initially, Subroutine POMPA was developed to site all lift stations (outlets), except the final outlet. Due to possibility of noise and odor problems, pumping stations cannot be placed arbitrarily. Suitable spots for lift stations may be selected prior to the generation of the layout. Therefore, this subroutine was modified to work with preselected pump locations as well.

4.2.3. SUBZONE SELECTION

To investigate the effect of the number of subzones to the performance of the LG algorithm, then network for PINARKENT, initially subzoned into 8, is redivided into smaller and larger subzones. For these runs, the program used the equivalents of 52811 words of storage capacity (21356 words for instruction bank and 31465 words for data bank). The CPU time was about 5 minutes. Results obtained are presented in Table 8.

TABLE 8. EFFECT OF NUMBER OF SUBZONES

Data	Number of Subzones	Σ Exc. Vol. (m^3)	Σ Depths (m)	Max Depth(m)	At Node
PINARKENT	GIVEN LAY	53 240	822.70	8.56	127
	12	58 886	887.79	9.31	188
	8	55 141	839.20	8.34	127
	6	52 565	807.95	8.24	127

The increase of excavation volume with the increase in the number of subzones has verified another intuitive expectation. Addition of one more subzone means repetition of the failures (in Section 4.1). Hence subzoning may lead to higher costs. It can be concluded that the network should be divided into as few subzones as possible. Since each subzone is considered as a single drainage area, the nodes to be contained in a subzone should be grouped considering not only their relative location but also the topography of the surface as well.

The efficiency of the LG algorithm is affected by proper selection of the subzones. A subzoned network is presented in Figure 16 and the essential aspects of this selection will be discussed here.

During the layout generation process, each subzone is considered as an individual drainage area with a known outlet. So, each suzone should be formed to conform with the natural topography. Every node in a subzone must be adjacent to at least one other node to be drained satisfactorily to the outlet.

To achieve the minimum number of subzones, the allowed capacity (here 70) must be fully utilized. Hence the number of nodes in each subzone should be as near to 70 as possible. It should not be forgotten that, the first

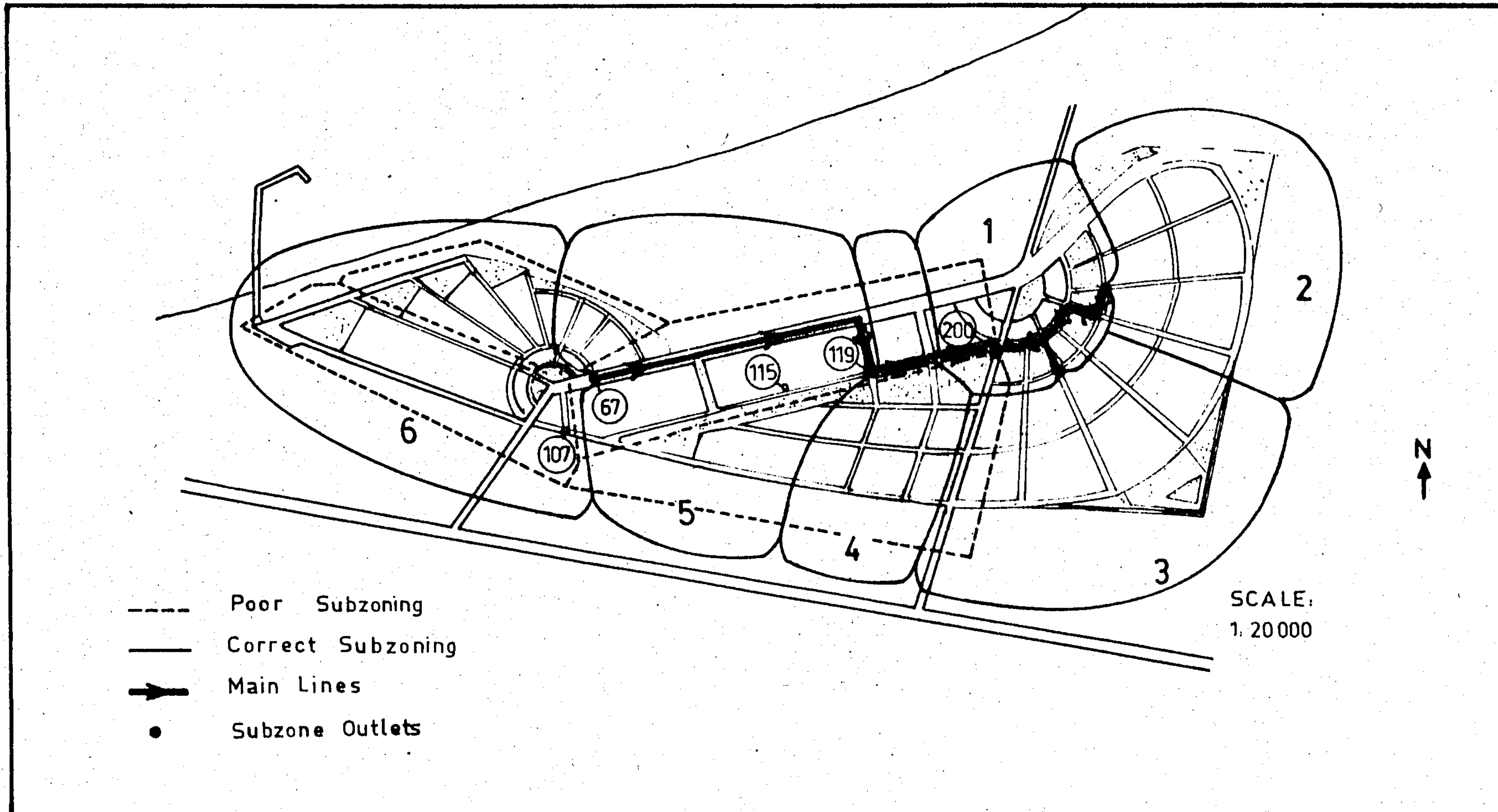


FIGURE 16. Subzoning of a Large Network (PINARKENT).

nodes of all links draining into a subzone are included into that subzone by the program. For example, Subzone 5 in Figure 16 initially had 60 nodes; for the number of nodes increased to 69. In Subzone 6, 62 increased only to 63. Therefore, the designer should either check such links or, considering possible additions stay at approximately 60 nodes.

The parallel orientation of adjacent subzones in the direction of the outlet should be avoided. To illustrate this a poor selection of subzones is presented in dotted lines in Figure 16. This recommendation is made to prevent parallel main lines, which are deeper than the laterals.

The order of analysis for the subzones must be from the closest subzone towards the fartherest. This recommendation is related with the order of execution and would result in some benefits.

4.2.4. LOCATION OF SUBZONE OUTLET

Since each subzone is considered as a single drainage area, its outlet should be selected at a suitable node, in accordance with its topography. The first two rows of Table 9 show the effect of such a selection. Here the selection of an appropriate subzone outlet yielded a better layout in that subzone.

TABLE 9. EFFECT OF SUBZONE OUTLET LOCATIONS

Data	Outlet Node for Subzone 6	Σ Exc. Vol. (m^3)	Σ Depths (m)	Max Depth(m)	At Node
Subzone 6 of PINARKENT	107	11 689	183.84	5.82	60
	67	10 222	178.27	4.75	55
PINARKENT (6 Subzones)	107	56 059	846.06	8.34	127
	67	52 565	807.95	8.24	127

On the other hand, the location of each subzone outlet must also be in agreement with the final outlet when considering the network as a whole. The last two rows of Table 9 present the effect of a change in a subzone outlet to the full network. Although the improvement in excavation volume, due to selection of a more suitable outlet location, is only 1467 m^3 (11689-10222) in Subzone 6, the reduction increased to 4268 m^3 (56833-52565) for the full network. This means that the selection of a subzone outlet does affect the excavation volumes of the adjacent zones as well. However, the opposite of this situation can also occur. Although a poor outlet selection increases the excavation volume of a particular subzone, it may decrease the total excavation volume of the whole network, meaning that it is a better subzone outlet.

4.2.5. LOCATION OF FINAL OUTLET

Location of the final outlet is also important for the total excavation volume of the full network. In all the runs of PINARKENT, the final outlet was selected at Node 200, as recommended by Alper et al.(1980). Here the effect of a different final outlet location, Node 119, is presented in Table 10.

TABLE 10. EFFECT OF FINAL OUTLET LOCATION

Data	Final Outlet Node	Σ Exc. Vol. (m^3)	Σ Depths (m)	Max Depth(m)	At Node
PINARKENT (6 Subzones)	200	52 565	807.95	8.24	127
	119	51 785	795.71	6.56	120

This change in final outlet location caused a 1.30% reduction in total excavation volume. The second final outlet location, Node 119, is selected from the nodes in the middle region of the full drainage area. Then, it can be concluded that the final outlet should be located somewhere in the central part of the drainage area to prevent too long main lines. This caution will decrease the invert elevations of the nodes crossed, which in turn, results in a decrease of the total excavation volume. Clearly, while locating the final outlet, the suitability of the node for practical purposes, such as treatment facilities, drainage to an existing system or river, sea, etc., and odor problems, should also be considered.

The Layout Generation algorithm is limited to function with a single final outlet. Therefore in case more than one final outlet exists in a network, there are two possible approaches to deal with such a situation: a) the network must be divded into two or more separate networks, each with a single final outlet, and designed separately, b) the final outlets may be connected to a hypothetical outlet by zero-length and zero-cost links.

CONCLUSIONS AND RECOMMENDATIONS

A computer program, SEWNET, developed for generation of the layout and hydraulic design of sanitary sewer networks, is presented in this study. Specification of the topographic conditions and the possible sewer lines in the drainage area are sufficient to run the program. Unlike the programs presented in the literature, the program does not need an initial network layout. Effect of the major hydraulic design parameters can be investigated by successive runs. Thus the major objective of the program, which was to decrease the role of engineering judgement in the overall design process, has been achieved to a considerable degree.

The hydraulic design algorithm presented here is an efficient one for the routine works encountered in the design of sanitary sewer systems with prescribed layouts. The layout may be described either in the form of a main layout or a full network. For a main layout, the algorithm connects the missing sewer lines to generate the full netowrk. The flow direction is selected so as to minimize the excavation for each sewer link. All hydraulic and topographic results obtained for each link are printed in tabular form for easy tracing. This form has been organized to conform with the Turkish practice (Iller Bankası, 1972a).

The Layout Generation (LG) algorithm uses a standard Shortest Path Algorithm along with a hypothetical excavation cost for each sewer to generate layouts with minimum excavation volumes. The hypothetical excavation for each sewer link is computed by laying the sewer at minimum cover depth and slope. Excavation minimization routine has been successful, particularly in medium sized networks. The high computer storage requirement for larger networks may be avoided by subzoning the drainage area and then superposing the layouts generated for each subzone.

The optimality level reached is somewhat restricted by the subzoning process. The excavations required decrease as the number of subzones is reduced. However, the efficiency of the algorithm can be improved by a successful selection of the algorithm can be improved by a successful selection of the subzones. Each subzone must form a single drainage area conforming with the local topography. Also, where additional computer storage is available, dividing the network into as large subzones as possible would lead to layouts with smaller excavations.

The siting of lift stations in the network may be done either manually or automatically by the program during the hydraulic design stage. For automatic siting, the program locates the lift stations at those nodes with depths exceeding an allowed depth limit. The concurrent assignment of preselected lift stations is also within the capabilities of the program.

The generated main layout can match the necessities of a storm sewer layout without any further modification since storm runoff is permitted to flow in street gutters over the unconnected links.

An extension of the present program would be the addition of an algorithm to modify the generated layout. This modification process should include the assignment of enough interconnections between subzones to result in lower excavation volumes. Likewise, assignment of a more realistic cost for the links in generating the Shortest Path Spanning Tree would generate solutions closer to the optimum. Recall that this cost must be in terms of the information known prior to layout selection. Specification of the "excavation cost versus depth" relationship, and the "variations of the soil type with corresponding excavation costs", would provide more accurate estimates for the actual excavation costs. The existing, EX criterion can be modified with such information to yield more econmical layouts.

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APPENDIX

USER'S GUIDE FOR SEWNET

1. INTRODUCTION

Use of computers for the design of sewer networks, makes possible, not only the analysis of many possible sewer layouts in a short time, but also the application of more advanced methods of analysis.

The hydraulic design of a sanitary sewer network with a prescribed layout is a routine but time consuming process when performed manually. Use of computer techniques reduces it to a relatively simple task.

The purpose of this manual is to introduce the computer program, SEWNET, developed at Boğaziçi University. This program accepts even the most general topography for a drainage area. It is capable to design networks with both prescribed and unprescribed layouts. For the latter case, first a layout is generated for the drainage area and then the design phase is completed. Here the capabilities of the SEWNET program is described and it is applied to the example worked in Section 3.2.3.1 to demonstrate its capabilities.

2. METHOD OF CALCULATION

The design of a sanitary sewer system can be divided into two phases: (1) determination of the system layout; and (2) determination of the pipe design parameters (elevations, slopes, diameters and manhole depths) for given layouts. These two together constitute the total design problem.

To achieve minimum-cost designs for sanitary sewer systems both phases of the problem should be considered simultaneously in an optimization procedure. But, due to the complexity of the problem, the present study is addressed to developing a computer program with particular emphasis on optimization of the system layout.

2.1. Hydraulic Design Computations

Various publications in sewer design, foremost of which is the ASCE Manual on "Design and Construction of Sanitary and Storm Sewers" (1969), discussed in detail the hydraulics of sewers, estimation of the quantity of sewage to be handled, and determination of the pipe design parameters. Here these are briefly summarized.

For the given drainage area, the design period and the quantity of sanitary sewage must be estimated. Assuming that the design discharge of an individual pipe is flowing at the half depth, Manning's Equation is solved for the pipe diameter. Then, with a known pipe diameter and design discharge the other hydraulic elements of an individual pipe is computed iteratively. If these are within the required specifications, then such topographic elements as invert elevations, slope and manhole depths are calculated using these values.

2.2. Layout Generation

The Shortest Path Spanning Tree drained to a specified outlet is determined for the given network and named as the generated layout. During this process, flow is permitted in the direction of natural slope.

To develop an optimization criterion, significant contributions to the total project cost are analyzed. Among these, excavation cost is seen to be the governing one and this minimization of the total excavation volume is taken as the sole optimization criterion. Since Shortest Path is a minimization algorithm an excavation cost is assigned to each link to achieve the overall minimum-cost layout (Section 3.2).

3. PROGRAM CONSTRAINTS

The hydraulic design part of the program is capable of handling a network with a maximum of 600 nodes. On the other hand, the layout generation algorithm can work with networks of a maximum of 70 nodes only. Both of these constraints are imposed by the limited capacity of the computer storage facilities. They can be modified to conform with the capacity of the machines used.

Layout generation, for networks having more than 70 nodes, may be achieved by subzoning a larger network. A subzone is delineated by specifying the nodes contained in it and by selecting one as the outlet node.

Here the layout for each subzone is generated separately. To satisfy continuity among the subzones, the outlet node of each subzone must also be specified among the nodes of the adjacent subzone to interconnect the two subzones. This, of course, is not necessary for the final outlet.

This multi-declaration of nodes in more than one subzone will increase the total number of nodes for the whole network. This increase must be equal to the number of subzones minus one.

4. DEFINITIONS

All input-output variables are defined below along with their typical values and restrictions, if any. All dimensional variables must be specified in SI units.

N = Total number of nodes ($\leq$ 600)

M	= Total number of possible links ($\leq$ 600)
MLAYOT=0	Flag for Layout Generation
MLAYOT=1	Flag for Given Main Layout
MLAYOT=2	Flag for Given Full Network
NNOUT	= Total number of outlets ($\leq$ 9)
NOUT(I)	= Array to store the number of the outlet nodes (I $\leq$ 9)
NNSUBZ	= Total number of subzones (=0 when running for the complete network)
NSUBZ(I)	= Number of nodes in each subzone (when running for the complete network)
DMIN	= Minimum Cover Depth (= 1.0 m)
SMIN	= Minimum Slope (= 0.0033 for Ø = 20 cm)
SINC	= Pipe Size Increment (= 10 cm for Ø > 30 cm)
RN	= Manning's n (= 0.013 for concrete)
CLOAD	= Flow coefficient at mean discharge (= $Q_{mean}/\Sigma_{pipe\ length}$ m^3/day/m)
VMIN	= Minimum velocity for full pipe flow (0.60 m/sec)
DEP	= Maximum manhole depth allowed (m; =0 if no restriction)
ELEV(I)	= Surface elevation for each node (m; I $\leq$ N)
HL(I)	= Horizontal length for each link (m; I $\leq$ M)
NODE(I,J)	= Initial (J=1) and terminal (J=2) nodes for each link (I) in the flow direction. (Flow direction is not important for MLAYOT=0; I$\leq$M)
NM	= Number of links in the given layout (= N-1)

LAYOUT(I) = Arrary for link numbers contained in the specified main layout ($I \leq NM$ declared only for MLAYOT=1)

NF(I) = Array for storing the node numbers in the corresponding subzone ($I \leq NSUBZ(J)$)

NUH = Outlet node number for the corresponding subzone.

Some of the above definitions belong to data variables, but some of them are control parameters. The second group will be re-explained here in detail.

The program first checks the value of MLAYOT. If MLAYOT=2, corresponding to a given full network, the program designs the directed complete network hydraulicly. If MLAYOT=1, corresponding to a given main layout, the program reads this main layout into LAYOUT($I \leq NM$). For both of the above cases variables NNSUBZ, NSUBZ, NF and NUH need not be specified.

If MLAYOT=0, this means that a layout will be generated for the network. At this step, the value of NNSUBZ is checked. NNSUBZ=0 again makes specification of variables NSUBZ, NF and NUH unnecessary. This means that the network is within the restrictions of the program capacity and that a layout can be generated at once.

If the network is divided into subzones, (NNSUBZ≠0), the total number of nodes in each subzone is declared into NSUBZ(I≤70). Then the corresponding node numbers are read into NF(I≤NSUBZ(J)) and the number of the Outlet Node into NUH. After the generation of the main layout for this subzone the necessary information is stored and the process is repeated for another subzone.

5. DATA PREPARATION

The user should follow the steps below for a successful run.

5.1. Network Labelling

Before describing the exact format of the data cards, a discussion of the labelling procedure for the network will be useful. Labelling steps are as follows:

a) At each junction or grade change, a node should be placed and numbered.

b) The surface elevations of each node should be determined.

c) All the possible connections between these nodes (simply possible links) should be determined and numbered.

d) The horizontal lengths of the possible links should be determined.

If the layout is to be generated by LG algorithm, these four steps are sufficient to run the program. If the network has more than 70 nodes, subzoning becomes necessary. Then:

e) Determine the following additional information
   i) Number of subzones (NNSUBZ)
   ii) Number of nodes in each subzone (NSUBZ(I))
   iii) Node numbers for each subzone
   iv) Outlet for each subzone
   v) Except for final outlet, outlet of each subzone must be declared again within adjacent subzone for interconnection.

In case either of the main layout or the full layout is to be manually generated and specified for the program, then:

f) The links forming the main layout and their flow directions must be specified. Flow directions for links which are not included in the main layout need not be determined.

g) For the full network, the flow directions for the unused links must be determined and these links must be connected to the main layout.

The procedure to be followed are summarized below for each of the above cases discussed:

a) For Layout Generation: Steps a,b,c,d,e

b) For Given Main Layout: Steps a,b,c,d,f

c) For Given Full Network: Steps a,b,c,d,f,g.

5.2. Program Data Cards

For simplicity in understanding, the input format will be given in their required order.

```
  READ{5,2}N,M,LAYOUT,NNOUT,{NOUT{I},I=1,NNOUT},NNSUBZ,
          {NSUBZ{J},J=1,NNSUBZ}
  READ{5,1}DMIN,SMIN,SINC,RN,CLOAD,VMIN,DEP
  READ{5,1}{ELEV{I},I=1,N}
  READ{5,1}{HL{I},I=1,M}
  READ{5,2}{{NODE{I,J},J=1,2},I=1,M}
  IF {MLAYOT-1} 3, 4, 5
5       CALL FULNET
        STOP
4       NM=N-1
        READ{5,2}{LAYOUT{I},I=1,NM}
        GO TO 6
3       DO 7 I=1,NNSUBZ
        NOT=NSUBZ{I}
        READ{5,2}{NF{J},J=1,NOT},NUH
7       CONTINUE
1       FORMAT{16F5.0}
2       FORMAT{16I5}
6    →  SPECIFIED MAIN LAYOUT{SML; Hydraulic Design Algorithm}
```

C INPUT DATA

```
1.        6      7      0      1      6
2.     1.0.0033  0.1  .013   0.5    60.
3.        5      4      3      2      3      2
4.       35     65     40     50     60     55     45
5.        1      2      2      3      1      4      2      5      3      6      4      5      5      6
```

C OUTPUT

NO. OF NODES	NO. OF LINKS	OUTLET NODE
6	7	6

ELEVATIONS

5.00	4.00	3.00	2.00	3.00	2.00

HORIZONTAL LENGTHS

35.00	65.00	40.00	50.00	60.00	55.00	45.00

GENERATED LAYOUT

LINK	N1	N2	LENGTH
1	1	2	35.00
4	2	5	50.00
5	3	6	60.00
6	4	5	55.00
7	5	6	45.00

DEPTH AT EACH NODE

1.00	1.00	1.00	1.00	2.18	1.33

KANAL NO.	B1	B2	L M	QB (	Q1 CM3/S	) Q2	ZK1 M	ZK2 M	D1 CM	D2 CM	TK1 M	TK2 M	EGIM M/M	CAP CM	HIZ D.	(CM/S) P.	M.	SU D. P.	(CM M.
1	1	2	35.0	26	0	26	5.00	4.00	100	100	3.80	2.80	.0286	20	176	23	26	.3	.4
2	2	3	65.0	49	0	49	4.00	3.00	100	100	2.80	1.80	.0154	20	130	22	25	.5	.6
3	1	4	40.0	30	0	30	5.00	2.00	100	100	3.80	.80	.0750	20	286	33	38	.3	.3
4	2	5	50.0	38	26	64	4.00	3.00	100	100	2.80	1.80	.0200	20	148	26	30	.6	.7
5	3	6	60.0	45	0	45	3.00	2.00	100	100	1.80	.80	.0167	20	135	22	25	.5	.6
6	4	5	55.0	41	30	71	2.00	3.00	100	218	.80	.62	.0033	20	60	14	16	.9	1.1
7	5	6	45.0	34	135	169	3.00	2.00	218	133	.62	.47	.0033	20	60	19	21	1.3	1.6

TOPLAM 20-LIK BORU UZUNLUGU= 350 M   TOPLAM KAZI= 333 M3   TOPLAM DERINLIK= 9.51 M

EN FAZLA DERINLIK= 2.18 M   BACA NO= 5