ABSTRACT

A VISUAL BASIC PROGRAM FOR THE DESIGN OF SEWER SYSTEMS

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The hydraulic design is one of the most important phases in sewer system projects. Conventional methods and classical engineering calculations for hydraulic computations consume more time and its ability of making mistakes is very high. Naturally, it is requested to be calculated in a short time with minimum mistake. A computer program is developed in this research, in Visual Basic 6.0 language which computes quantity of wastewater with respect to population, diameter of pipe, slope of line and installation elevations completing with cost estimations regarding the optimization techniques. The main objective of the program is to present the possible combinations of pipe diameter and line slope and then to choose the best alternative among them regarding the economic evaluation. This is principle, however more details can be added to improve the program for more accurate calculations.

Key words: Population estimation, sewer system, sewer discharge, pipe diameter, line slope, velocity, cost, optimization.

ÖZ

KANALİZASYON SİSTEMLERİNİN TASARIMI İÇİN VISUAL BASIC İLE BİR BİLGİSAYAR PROGRAMI HAZIRLANMASI

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Hidrolik tasarım kanalizasyon sistemi projelerinde en önemli evrelerden biridir. Hidrolik tasarımındaki işlemler geleneksel metotlar ve klasik mühendislik hesapları ile uzun zaman alır ve bunlarla hata yapma olasılığı yüksektir. Doğal olarak daha kısa zamanda ve en az hatayla hesaplanması arzu edilir. Bu çalışmada, optimizasyon tekniğiyle maliyeti sonlandıran, nüfusa göre pis su miktarını hesaplayan, boru çapını, eğimi ve uygulama yükseltilerini bulan Visual Basic 6.0 diliyle yazılımı yapılmış bir bilgisayar programı geliştirilmiştir. Bu programın temel amacı mümkün olan boru çapı ve eğim kombinasyonlarını sunmak ve ekonomik değerlendirme yaparak bunların arasından en iyi alternatifi seçmektir. Bu prensiptir ama programın daha doğru hesaplamaları yapması için geliştirerek fazla detaylar eklenebilir.

Anahtar kelimeler: Nüfus tahmini, kanalizasyon sistemi, pis su debisi, boru çapı, eğim, hız, maliyet, optimizasyon.

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1. INTRODUCTION

Starting from the early civilizations which developed around bodies of water, people have supplied water from various resources and disposed of that used water in anyway. As being need to find and use water for life continuity, there is as much as need to remove that water for being a civilized population.

Water has been playing an important role in living's life since the beginning of world history. Human used water for various purposes in different fields such as domestic needs, irrigation needs, industrial needs and in many other civilization requirements. These compulsory water consuming forced to improve a process called *sewerage* which refers to the collection, transmission, treatment and disposal of liquid waste.

The sewerage process includes phases as: hydraulic design, installation and operating methods all of which need engineering experience, initiation and knowledge to achieve and serve successful projects for populations to be civilized.

The purpose of making hydraulic calculations for sewer system is to obtain: sewer discharge, transmission dimension (pipe diameter), line slope, and installation elevations in order to contain maximum flow without being surcharged by maintaining velocities which will prevent deposition of solids. The major aim in these calculations is to present an adequate solution which provides economic and long serviceable results regarding of inputs and constraints. Naturally, it takes too much time to obtain those values in the direction of the major aim by using manual methods and hand calculations due to abundance of variables and alternative combinations of solutions.

In this study, the possible combinations of those two variables; pipe diameter and slope, is investigated to obtain the best combination which refers the reduced excavation amount with the economic pipe section. The study is prepared as a computer program using *Visual Basic 6.0*, having the alternative of changing variables easily on the basis of some constraints and presents the possible combinations with their costs by using the optimization techniques.

2. WASTE WATER COLLECTION AND REMOVAL

2.1. Design Objectives

The design of a gravity wastewater collection system must provide an engineered system of sewers, complete with all appurtenant facilities, sufficient in size, slope and capacity to collect and convey the required wastewater flows to an acceptable point of discharge. [1]

The system must be practicable, economically feasible, and must be located to minimize the costs of installation, operation and maintenance. Sewers and appurtenances must be structurally sound and must protect the environment from pollution caused by leakage at pipe joints or manhole structures. Extraneous flows that hydraulically overload the system and produce flooding at sewer manholes and lift stations must be excluded. [1]

Elimination of excessive infiltration and inflow is essential in avoiding increased costs of sewer maintenance, wastewater pumping and treatment. Even more important in this regard is the necessity to maintain design wastewater treatment efficiencies, and thus assure that effluent discharge requirements are met. Contributing waste flows which are harmful to sewer pipe materials are appurtenant structures, toxic to biological and other waste treatment systems, or create fire and explosion hazards, must be identified and evaluated early during pre-design, so that suitable materials and/or procedures for their disposal can be included. [1]

Design constraints, appropriations and calculation techniques may vary for many countries in accordance with the type of materials used, government restrictions, topographic conditions, budgets and factors like these. Despite the abundance of these design parameters, the main objective intersects at the same point and becomes unique: to create a system that serves with minimum operating and cost problems during the scheduled project life, naturally constructed in an estimated limits of budget regarding environmental and the health requirements.

2.2. Limitations on Use

To protect sewers, pumping stations and treatment facilities from unwanted pollutants and extraneous flows that result in excessive operation and maintenance, fire and explosion hazards, or reduced wastewater treatment efficiencies, limitations must be placed on the use of the sewer system. [1]

Wastewaters from fuel loading and dispensing systems, grease and oil from vehicle wash racks, mechanical equipment washing and garage or shop floor drains, must be directed through oil/water separators to prevent such wastes from entering the sewers. Combined sewers should not be permitted, and collection of storm drainage of any kind must be avoided. All types of industrial wastes must be analyzed to determine if any substance is detrimental to sewer pipe materials, waster treatment processes, or creates a safety hazard to people or other living things. [1]

2.3. Selection of Sewer System

Sewage is transmitted in a closed conduit called as a sewer, which normally flows partially filled. In conventional practice, the domestic and public sewage, industrial wastes and the storm runoff are all collected and transmitted in a common system known as a combined system. Since the capacity of the combined system is not enough to carry a heavy storm runoff having a considerable return period together with the other waste water components, it is recommended to construct a separate system in which two different systems carry the sanitary sewage and the storm runoff separately. [2]

As urbanization increases, the storm runoff increases due to higher imperviousness that characterizes urban areas. Therefore, the waster water collection systems of the large cities are to be of a separate system. [2]

Both of the systems have advantages and disadvantages according to each other. The conditions of the design area and the existing system should be investigated well to obtain an economic solution and then the system selection can be performed. Some part of the existing system may be used as storm water system and some part may be used combined or the new system may serve as separate to get the most economic solution. Whenever the existing system is not sufficient for an adequate use, then a new sewer system is planned and designed. [3, 4]

Needing special sections in dimensioning, having settlement problems, causing plugging, overflows in heavy rains and like for similar disadvantages, separate system sewers are preferred and designed. In our country also the main related establishment *The Bank of Provinces* recommends and designs separate systems. The system selection is done better by economic comparison of systems which of them have the chance to carry out the objective with technical requirements. [3]

2.4. Types of Gravity Sewers

The principal types of gravity sewers that make up a wastewater collection system, starting with the smallest and proceeding to the largest, may be described as follows: (1) *building sewers* are used to connect the building plumbing to a lateral or branch sewer; (2) *lateral* or *branch* sewers, which form the upper ends of a wastewater collection system and are usually located in the streets or special easements, are used to convey wastewater from building sewers to main sewer; (3) *main sewers* are used to convey wastewater from one or more lateral sewers to trunk sewers; (4) *trunk sewers* are large sewers used to convey wastewater from main sewers to the treatment facilities or to large intercepting sewers; (5) *intercepting sewers* are very large sewers used to intercept and convey the flow several trunk sewers to treatment or other processing facilities. [4]

2.5. Liability for Damages Caused by Sewage

A city cannot be required to furnish sewerage, nor can it be held liabile if it is not furnished. Once such service is provided, however the community and its officials assume certain responsibilities for damages to health, property, or the environment which may result from unsatisfactory design construction, or operation of system. [5]

Deficiencies in design or construction may permit the city to involve the engineer or contractor in any legal difficulties which result. The city however, as owner of the facilities may bear the ultimate responsibility if the other parties are unable to pay for the damages. [5]

Damages resulting from poor operation or maintenance are always the responsibility of the city. Failure to respond quickly to known deficiencies is normally sufficient to establish legal liability. [5]

The liability is a very important thing for such these infrastructure constructions which are serving directly for the population. Sewer systems can not be seen visually during serving as being buried underground, unfortunately the system may be a killer if the necessary technical specifications are not applied in the performance of the all execution phases. The contribution of hazardous chemicals to water supply system is one of the most common situations because of the deficiencies probably formed in the design phase and/or construction phase.

Wherever human kind exists, human errors will. In spite of this fact, the main aim is always towards decreasing the number and severity of human errors. For several financial and educational reasons, the human-related errors made in sewer systems are encountered more frequently in developing countries rather than in the developed ones. The already existing financial difficulties, health problems, and inconvenience in the daily life are usually increased with the occurrence of errors in the sewer systems in developing countries. [6]

Minimizing the need for sewer systems repair and renewal can be achieved only through careful planning, design and operation. However, this objective is rarely achieved. The responsibility of the quality of the sewer systems can be related to three main parties: the government; the sector/s responsible for the design, operation and maintenance of the project; and the citizens. The government in its wide definition is certainly considered to be the main coordinator, and it takes most of the responsibility of the success of failure of the project. This is due to having the power to control all the performance of the relevant sector/s involved in the project, and also, due to having the power to educate citizens about the proper use of sewer systems and punish the ones who might cause any damage. [6]

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3. PRELIMINARY DESIGN CONSIDERATIONS

3.1. Preliminary Investigations

As an important initial step in the design process, existing maps, drawings, surveys, boring logs, and other data containing pertinent information on existing conditions in the area being sewered must be obtained. [1]

Preliminary investigations provide a basis for cost estimates which are used to evaluate the feasibility of a project and to justify bond issues, assessments against property, or the other methods of fund raising. [5]

Fairly detailed maps are available for most communities. Towns which do not maintain official maps may have been mapped by assessors, insurance companies, or public utilities what will normally permit their map to be copied. If no map are available, aerial strip photography is probably the least expensive method of obtaining a map with the necessary detail. [5]

Preliminary designs are based on estimated flows, approximate ground contours, the location of the streets or sewer easements, and the location or locations to which the sewage is to be taken. These preliminary designs will permit estimation of the quantity of pipe of various sizes, the quantity of excavation, the quantity of pavement repair, and the various appurtenances which will be required. [5]

Cost estimates are made for the alternatives identified as being physically practicable and environmentally acceptable. Costs should be based, when possible, on bid tabulations for recent similar construction. When such information is not available, one may use national averages corrected to correspond to the local cost index. [5]

3.2. Detailed Design Requirements

Before final lines and grades are established for a sewer system, an underground survey should be conducted to establish the location of existing sewers; water and gas lines; electrical, telephone, and television wires; foundations; and other construction which might present obstacles to the proposed design. Many city engineering departments maintain maps showing all underground structures. When such city maps are not available, the designer must compile the information from the various utility companies. [5]

The presence of rock or other difficult subsurface condition in the construction area will have a significant effect on costs; hence soil borings or soundings are desirable. Soundings may be made by driving a sharpened steel rod into the ground until rock is struck. For depths in excess of 5 to 6 m, borings are necessary. The number of soundings or borings should be sufficient to establish the location of the rock surface throughout the area of the project, and will thus depend the geological character of the area. [5]

Preparation of construction drawings requires knowledge of street pavement types, the location of all underground structures, the location and basement elevations of all buildings (elevations are usually estimated for residences), the profiles of all streets in which sewers are to be placed, and the elevations of the maximum water surface and invert of all streams, culverts and ditches. Permanent benchmark should be established during the survey for use during construction. A detailed map must be prepared, on which the information is listed above, together with ground contours, elevations of street intersections, and any abrupt changes in street grade are noted. The scale of map is typically 1:500 to 1:1000 with a contour interval of about 300 mm. [5]

A tentative layout of the proposed system is made by locating lines along the streets or utility easements with arrows showing the direction of flow normally in the direction of ground slope. The result will be a main sewer leaving the area at its lowest point with submains and laterals radiating to outlying areas and following the natural slope of the ground to the extent possible. Ridges within the area served may require construction of systems with separate discharges or pumping across the high area. In flat terrain, all sewers may be sloped to a common point from which the collected flow is pumped. [5]

Location of sewers in backlot utility easements will minimize damage to pavement but makes access somewhat more difficult. Generally water, gas and sewer lines are placed within the street right of way. It is desirable that water lines and sanitary sewers be separated, preferable by the width of the street. On very wide streets, sewers may be placed on both sides to reduce the length of the household sewers. [5]

The vertical location is limited by the need to provide minimum cover and service to basement sanitary facilities and the desirability of minimum excavation. In northern states, 3 m of cover may be required to prevent freezing. In southern states, minimum cover is dictated by traffic loads and ranges upward from 0, 75 m, depending on the pipe size and the anticipated loads. [5]

Manholes are located at sewer intersections, abrupt changes in horizontal direction or slope, changes in size, and at regular intervals along straight runs. Manhole spacing generally does not exceed 100 m and should never be greater than 150 m except which can be walked through. Each manhole is numbered and the numbers of the manholes also serve to identify the sewers, which run from manhole to manhole. [5]

The area tributary to each manhole is sketched on the map, on the basis of ground contours and the locations of lots and buildings. Every manhole need not have a tributary area. For storm sewers a similar procedure is used except the lines are considered to run from inlet to inlet or from intersection to intersection of the streets. [5]

The vertical profile is prepared for each sewer line at a horizontal scale of 1:500 to 1:1000 and a vertical scale about 10 times greater. The profile shows the ground or street surface, tentative manhole locations, elevation of important subsurface strata such as rock, locations of borings, all underground structures, basement elevations, and cross streets. A plan of the line and relevant other structures is usually shown on the same sheet. [5]

3.2.1. Field investigations

If maps are not available or do not provide satisfactory information or sufficient detail of the site, field surveys must be performed. Depending on the magnitude and complexity of the project, subsurface exploration with soil borings may be required.

Topographic information should show locations of all streets, buildings, pavements, sidewalks, vegetation, drainage channels, and the other surface features such as utility manholes or structures, which may influence the design and layout of the collection system. Information on existing utilities should include the location of underground water lines, sanitary sewers, storm drains, gas mains, electric conduits and similar facilities. For larger projects, the character of the soil in which sewers will be constructed should be determined. The presence of rock, unfavorable soil conditions, or high groundwater table should be clearly established. [1]

3.2.2. Guidelines for sewer system layout

The development of final sewer plans must await the final site plan, the completion of field surveys, and to some extent, the establishment of floor grades. However, the development of economical site plans often requires concurrent preliminary planning of the sewer system. The location of the building and lateral sewers will depend not only upon topography, but also upon the type and layout of the buildings to be served. Normally, the most practical location would be along one side of the street. In other cases they may be located behind the buildings midway between streets. [1]

In still other cases, in closely built-up areas and particularly where the street is very wide or already paved, it may be advantageous and economical to construct laterals on each side of the street. Main, trunk, and interceptor sewers will follow the most feasible route to the point of discharge. All sewers will be located outside of roadways as much as practicable, so that the number of roadway crossing will be reduced to a minimum. A sewer from one building will not be constructed under another building, or remain in service where a building is subsequently constructed over it, if any other practical location for the sewer is available. Where no other location is suitable, necessary measures will be taken to assure accessibility for future excavation and complete freedom of the sewer from superimposed building loads. [1]

The following general criteria will be used where possible to provide a layout which is practical, economical and meets hydraulic requirements:

- Follow slopes of natural topography for gravity sewers.
- Check existing maps or field surveys along prospective sewer routes to assure that adequate slopes are available.
- Avoid routing sewers through heavily wooded areas and areas which require extensive restoration after construction.
- Check subsurface investigations for groundwater levels and types of subsoil encountered. If possible, avoid areas of high groundwater and the placement of sewers below the groundwater table.
- Locate manholes at change in direction, size or slope of gravity sewers.
- Sewer sections between manholes should be straight.
- Manholes should be located at intersections of streets when possible.
- Avoid placing manholes where the tops will be submerged or subject to surface water inflow.
- Designer should evaluate alternative sewer routes where applicable
- Verify that final routing selected is the most cost effective alternative that meets service requirements. [1]

3.2.3. Population data and average demand

The amount of used water is the main input value for the design of sewer system. This amount can be estimated after some estimations regarding population growth, water demand, fluctuations in water use, return period, and percentage used. These variables are used as a parameter during the calculation of sewer discharge.

Before the determination of water used, naturally the amount of water to be used should be calculated and then the all of it or the percentage of it can be taken as sanitary sewage flow. The amount of water to be used according to the population is obtained from municipal or other governmental establishments which are present in their specifications. In this study, the water demand values are obtained from the specifications of The Turkish Bank of Provinces.

Municipal water demand is the combination of requirements for domestic, public, commercial and industrial uses and fire fighting. Domestic use includes the water demand for drinking and sanitary purposes. Population size and living standards affect the domestic use. Therefore, it is a better approach to express the water demand on a per capita basis, which reflects the differences in water uses with respect to districts. Domestic water consumption per capita may reach about 1000 lt/cap/day in developed countries. Public use is the water is the water requirement in public establishments, schools hospitals parks, etc. Water use in public establishments is normally required within certain hours during a day. Therefore, public use is generally lower than the domestic use. [2]

Water supply systems are designed to meet total projected municipal demand. The preliminary step may be the estimation of the life time of the system by considering the type of the system, the desired degree of hydraulic conformity, availability of technology, materials appurtenances and economy. The next step is then the estimation of the projected water requirement corresponding to the end of the life time of the system, which is normally based on the previous records concerning the estimations for the population and growth rates in the city under study. [2]

In the design of sewer systems, the projected population (estimated future population, which refers EFP in *SewDes 1.0*) of the community at the end of the life time of the project is to be estimated. Topographic and climatic conditions of the community and the socio-economic facilities available in the city may affect the rate of migration, and hence the population growth. Before selecting the suitable method for the population estimation of a community concerned, the past records of census results and the socio-economic developments in the region should be investigated.

According to the nature of the rate of population growth and characteristics of the community, the best method can be selected among the followings listed below: [2]

- Arithmetic extrapolation
- Geometric (logarithmic) extrapolation
- The Turkish Bank of Provinces method
- Mathematical curve fitting method;
- Logistic S-curve fitting method;
- Decreasing rate of increase method;

- Ration and correlation method;
- Component method; and
- Employment forecasts

In *SewDes 1.0* The Turkish Bank of Provinces method is used to estimate the future population. The formula and the necessary information exist in the program and can be reached easily by one click on related button.

4. SEWAGE FLOW

4.1. Population and Population Distribution

The amount of sewage flow as being directly in relation with the population distribution, the necessary detailed research for the past and future population should be done to plan a scientific and suitable comprehensive sewer system for a city. Population estimation researches and studies provide major data for the quantity of sewage flows in future and amount of organic materials that will be dispatched. [7]

The sewer system is designed by using the discharge which is obtained according to the estimated population for a convenient future. Graded improvement can be applied if there is a large difference between the existing and estimated population or if there occurs a compulsion of investments on the financial power. This improvement may be applied when the vision and opinion is suitable regarding the engineering and financial aspects. [7]

4.2. Estimation of Domestic Wastewater

The quantity of domestic wastewater from an area will generally be about 60 to 85 percent of the water supplied to the area. The remainder is used in industrial processes, for lawn sprinkling, etc. Hence, if the water use of a community is known, the probable output of domestic wastewater can be estimated. Estimates for wastewater facilities should allow for future growth of the area. [4]

Estimates of water supply must include all water from private as well as public sources. Industries often obtain water from their own wells but use public sewers for waste disposal. In this case combined industrial and domestic wastewater may exceed the water supplied by the public system. Conversely, some industries that draw water from the public supply may not discharge their waste into the public sewers, and this results in a low ratio of wastewater to water supplied. A careful study of local conditions is necessary for an accurate estimate of wastewater flow. [4]

The sewer systems in Turkey are designed according to the specifications of the "*The Bank of Provinces"* The related protocols are Protocol-I and Protocol-II in the sewer design and planning specification. *SewDes 1.0* uses these Protocols and determines the unit sewer discharge for hydraulic design stage.

4.2.1. Estimation of future population (Population Forecasting)

The water supply and sewerage projects are designed to meet the requirements of the future population generally further on 30 or 35 years. For this reason, the estimation of future population is required. In order to calculate the future population, the previous populations are examined. The population fluctuations and the probability of future growth and development are considered during estimation. [3]

Any water supply system must be planned to serve the present as well as the future needs of the community. Therefore future population must be assessed while designing the water supply. This requires foresight and judgment. The population changes due to;

- Births
- Deaths
- Migration

Birth, death, migration rates are dependent on many factors. It is up to the forecaster to judge which factors are to be considered in predicting the future population. [9]

Factors affecting the forecast of population are:

- The Period of forecast (when the period increases, the accuracy decreases)
- The population of the area (when the population decreases the accuracy decreases)
- The rate of increase of population (when the rate increases, the accuracy decreases)

In fact it is not possible to forecast population definitely, because deaths and births can not be estimated without any error and also migration increase ratios or decrease ratios can change exponentially. Despite some errors it is the best way to estimate a number at safe side, without estimation of population values; it is the worst case to construct a system that will work for 30 years or more.

In *the Bank of Provinces method*, to estimate the population, the *population increase coefficient* (which refers to PIP in *SewDes 1.0*) is to be calculated. The population increase coefficient refers to the population increase percentage and expressed as “p” in the specification of *The Bank of Provinces.* This percentage which is to be accepted according to growth circumstance is linked to the basis below:

If p is smaller than one ($p<1$)	then; p is one ($p=1$)
If p is greater than one ($p>3$)	then; p is three ($p=3$)
If p is smaller than one and greater than three ($1<p<3$)	then; p is one ($p=p$)

Where there occurs a great difference in the population due to season fluctuations, this case is considered as separate and mentioned in the demand calculations. [3]

In the case of an expected extraordinary population increase due to military, industry and tourism or any similar circumstances, there should be an agreement with the management establishment. According to these fundamental principles, the population increase percentage and the future population are calculated [3]

Population increase percentage (which refers to PIP in SewDes 1.0) is calculated by the equation below:

$$P = \left(\sqrt[a]{\frac{N_y}{N_e}} - 1 \right) \times 100 \tag{4.1}$$

here;

P : Population increase percentage

N_y : New population (Last census)

N_e : Previous population (Population of any past census)

a : Difference of years between new and previous census

Estimated future population (which refers to EFP in SewDes 1.0) is calculated by the equation below:

$$N_g = N_y \times \left(1 + \frac{P}{100}\right)^{30+5+n} \tag{4.2}$$

here;

N_g : Population after the 30+5+n years later from last census

N_y : New population (Last census)

P : Population increase percentage

n : Difference of years between the last census and the project year

As an example, the calculated population increase coefficients of Gaziantep are given in table below. [9]

Table 4.1 Population increase coefficients of Gaziantep

Years	*Population increase coefficients*
1985-1945	5.21
1985-1950	5.57
1985-1955	5.48
1985-1960	5.55
1985-1965	5.56
1985-1970	5.08
1985-1975	4.75
1985-1980	5.34

The population forecasting values that are found by this method are given in the Table below: [9]

Table 4.2   Population forecasting values for Gaziantep

Forecasting year	*Population value*
1990	596.500
1995	743.000
2000	926.000
2005	1.154.000
2010	1.438.500
2015	1.793.000
2020	2.234.000
2022	2.439.500

4.2.2. Calculation of water demand in the future

After calculating the estimated population in the future, the water demand of this population should be calculated. Water demand for one person is depended on the population of settlement region, climate, living standards, schedule of rates and many similar factors. It is not possible to compute the amount of affect of these factors on water demand. In this case, the best way is to determine a demand value which is the closest one to the real water demand value. The *daily water demand* of one person is determined according to the specifications of The Bank of Provinces with respect to population. In this case, the water demand value increases well-proportioned to population. [3]

The daily water demand values in a settlement region (generally a city) with respect to population, is given in the Specification-III of The Bank of Provinces.

Table 4.3 Average total demand with respect to population

Population	*Average demand (lt/cap/day)*	*Average demand (lt/s)*
3000 or less	60	2,1
3000 - 5000	70	4,1
5000 - 10000	80	9,2
10000 - 30000	100	34,7
30000 - 50000	120	69,4
50000 - 100000	170	196,8
100000 - 500000	230	1330,0
500000 - 1000000	280	3240,0
1000000 – 2000000	330	7640,0
2000000 - 3000000	370	12850,0

Interpolation is made for the populations which have the values among the ones in the table above. In fact, average demand values for the populations which are bigger than 100.000 (A hundred thousand) are decided together with the management of The Bank of Provinces. [8] However, the demand values for higher populations are mentioned as the values listed in the Table above. [2]

The water demand values in the related table are the *average daily demand* values for one person. Because of the changing of consumptions due to changing conditions such as climate, living standard and development of the city; the demand value is multiplied with a maximizing coefficient (which refers to c). This coefficient is generally taken as 1, 5 for summer days.

Certain dry years cause more consumption. In hot months more water is consumed in drinking, bathing and watering lawns and gardens. On holidays and weekends the water consumption may be high. Even during a day water use varies with high use during morning hours and close to noon and low use at night. Generally in smaller communities the variation is large. In bigger cities and towns the demand for water tends to be close to the average. [9]

Beside the usage of this table above, also some special water demand values can be determined from realistic analysis. If the population does not match the intervals in the table or if a special design is required; the demand values for these requirements are listed and the demands can be added to determine the daily eater demand value. In *SewDes 1.0* an alternative way for this type of calculation is present. This manual demand input event enables the user to use his/her computed demand value for design calculations. The table below may also help the user as a guide to determine the special demand values.

Table 4.4 Some of the water demand values for special requirements

Type of use	*Requirement*
Washing of cars (one time)	200-300 lt
A student in a school	2-10 lt/day
A soldier in military barracks	50-150 lt/day
A patient in a hospital	250-600 lt/day
A client in a hotel	100-250 lt/day
A bath in a Turkish bath	300-350 lt/day
A pool (for one square meter)	500 lt/day

The affects on consumptions may vary due to many factors which of some are mentioned in the previous paragraph. Considering this variation, it is necessary to have the ability to use the coefficient according to decisions of design engineers. *SewDes 1.0* has the chance to select this maximizing coefficient (c) starting from one and ending at two.

In hydraulic design calculations, the unit for water amount used is: lt/s. In this case, the calculated demand values are converted to this unit with the formula below:

$$Q_{DFP} = \frac{N_g \times Q_1}{86400} \tag{4.3}$$

here;

Q_{DFP}	: Average daily water demand value of future population	(lt/s)
N_g	: Estimated future population	(person)
Q_1	: Average daily water demand	(lt/cap/day)
86400	: Total seconds in one day	(seconds)

Average daily water demand can be also calculated by the designer for special circumstances by using minimum and maximum water consumption values which are determined and published by related establishments or experienced designers.

4.2.3. Calculation of unit sewer discharge

The amount of water calculated as Q_{DFP} is the most important flow value that forms the major place in the hydraulic design calculations. [3]

All of the water used does not return to sewer; some of it evaporates, contributes to ground during watering gardens and some usages like these. The ratio of the amount of the water return to sewer to the supplied water is shown as P_1 coefficient (refers to M value in *SewDes 1.0*). If there is not any special situation in order to be at the safe side, it is considered as all of the supplied water returns to sewer. In this case P_1 coefficient is taken as one. [3]

Now it is clear that not all of the municipal supply returns to the sewer system. Some is lost in the network and some is wasted lawn sprinkling, street washing, car washing, etc. Therefore the amount of sanitary sewage may vary from 70 to 130% of the average daily water consumption. The design waste water discharge is estimated on the basis of the per capita use corresponding to the end of the life time of the system. The contributions of the groundwater and rainfall to the system are also considered in the design. Since the amount of sewage is a function of daily water consumption, it shows fluctuations throughout a day. However, these variations are not as pronounced as those in water supply system because partially filled sewers provide a degree of equalization. [2]

The quantity of waste water return to sewer may be taken as the same quantity supplied by the municipal system which refers to use P_1 coefficient as one. *The Bank of Provinces* Method uses this coefficient as one in design calculations. In *SewDes*

1.0 there are two alternatives for this usage. The user may select the specification requirements or his/her calculation percentage as return coefficient.

The quantity of supplied water in a day returns to the sewer system in a shorter time period then a day interval. This case is defined as P_2 coefficient (Refers to N value in *SewDes 1.0*). This coefficient is determined according the basic principle that the municipal supplied water in a city in 24 hours, returns to the sewer system in T_d hours time period. [3]

Usually it is accepted that, the quantity of municipal supplied water returns to the sewer system in 12 hours (Specifications of The Bank of Provinces). In this way;

$$P_2 = \frac{24}{T_d} \qquad ; T_d = 12 \tag{4.4}$$

$$P_2 = \frac{24}{12} \qquad ; P_2 = 2$$

The calculated ($P_2 = 2$) value is used in design calculations of The Bank of Provinces Method. This coefficient is taken as 8 hours in villages and 16 hours in cities. [3]

The quantity of water used is calculated with these coefficients. The amount of water supplied, the percentage of water used and the return time to sewer system are all known, hence:

$$Q_{used} = P_1 \times P_2 \times Q_{DFP} \times c \tag{4.5}$$

here;

Q_{used} : The amount of water flow return to sewer system (lt/s)

P_1 : Return percentage to sewer system

P_2 : Return period to sewer system

Q_{DFP} : Average daily water demand value of future population. (lt/s)

c : Maximizing coefficient

From now on the amount of water flow return to sewer system (Q_{used}) can be called as the *sewage flow*. This sewage flow has to be calculated for relevant populations and has to be converted to unit value for hydraulic design calculations.

The unit sewer discharge for relevant populations living in tributary areas is calculated by the formula which is given in Protocol-1 of Specification *of The Bank of Provinces*.

In Protocol-1 of Specification; in a city with N amount of population with Q quantity of water consumption per capita per day, the total sewer discharge is:

$$q = \frac{N \times Q}{12 \times 3600} \quad \text{(lt/s)} \tag{4.6}$$

For different regions having areas F1, F2, F3…with the populations N1, N2, N3…and with the total lengths of sewer lines in these regions L1, L2, L3…, then the unit sewer discharges will be calculated as below:

$$q1 = \frac{q \times N1}{N \times L1} \qquad q2 = \frac{q \times N2}{N \times L2} \qquad q3 = \frac{q \times N3}{N \times L3} \qquad \text{(lt/s.m)} \tag{4.7}$$

In this way the unit sewer discharges for different density regions are determined for in the first preliminary study. Afterwards, the sewer discharges for each pipe during the arrangements of projects; can be determined by multiplying the unit sewer discharge with the relevant lengths of pipe lines. [8]

The unit sewer discharge can also be determined easily with one formula which is the combination of those discharge formulas above. *SewDes 1.0* uses that type of formula as:

$$Q_{sewer} = \frac{P \times W \times M}{N \times L \times 3600} \quad \text{(lt/s.m)} \tag{4.8}$$

here;

Q_{Sewer} : Unit sewer discharge. (lt/s.m)

P : Population living in design area. (Person)

W : Maximized water demand. (lt/cap/day)

M : Return percentage to sewer system. (%)

N : Return time to sewer system. (Hour)

L : Total length of sewer line in design area. (Length meter)

3600 : Total seconds in one hour

Finally, the design discharge is calculated by one more equation. The design discharge changes from line to line whit respect to length of sewer line. The relative interval between two successive manholes forms the variable to determine the design discharge. The formula is expressed as below:

$$Q_{desdis} = Q_{Sewer} \times L \qquad (lt/s) \tag{4.9}$$

here;

Q_{desdis} : The sewer design discharge (lt/s)

L : Length of sewer line to be designed. (m)

The design discharge calculated above is not only the value to be considered in hydraulic computations. The industrial discharges, storm discharges and special discharges are also considered together in sewer system designs. But the aim of this study is just for sanitary sewer design so; the industrial and storm discharges are not taken in consideration during the calculation of design discharge.

Despite its being a sanitary sewer design, the contributions of the groundwater and rainfall to the system are also considered in the design. These are handled not as storm water. The leakage of rainfall drops into manholes and infiltration of groundwater to sewer lines can not be neglected so; input boxes for these water quantities are added to *SewDes 1.0* as additional discharges.

The units of input values for these additional discharges are in lt/s for each design line. Usage of this addition is explained in part *SewDes 1.0* manual. The values for additional discharges can be taken from the Project Reports such as Feasibility Reports or Preliminary Research Reports.

5. PLANNING AND HYDRAULIC DESIGN OF SEWERS

5.1. Planning and Layout

Planning for the economical development of a sewer system requires information on current flows and forecasts of future flows. The projection of flow increases should provide sufficient lead time to formulate economic proposals, secure approvals, arrange financing, design, construct and place in operation the necessary sewers to carry domestic, commercial and industrial wastewater from a community to a point of treatment. [10]

A design period must be chosen and sewer capacity planned that will be adequate. Professional planners are reluctant to predict land use or population changes for more than 20 years into the future. However, when planning, design, financing and construction are considered together with the relatively minor additional cost of providing extra capacity, a 30 year design period is the minimum that should be considered. Planners should design for ultimate development where special conditions exist such as remote areas near the boundary of a drainage area. Also to be considered are areas where special construction, such as tunnels and siphons, may be required. The cost of additional capacity is minimal compared to the cost of relief lines installed at a later date. [10]

Mainline sewers should be designed for the population density expected in the areas served, since the quantity of domestic sewage is a function of the population and of water consumption. Trunk and interceptor sewers should be designed for the tributary areas, land use and the projected population. For these larger sewers, past and future trends in population, water use and sewage flows must be considered. The life expectancy of the pipe is critical when considering extended design periods. [10]

A good map of the area to be sewered is essential. Tentative routes for the main or trunk sewer and its principal tributaries are first selected. If possible, all sewers are planned to slope in the same direction as the ground surface. The layout is governed largely by the topography and the distribution of population and industry. Several trial layouts may be necessary before the most economic arrangement of sewer can be selected. [4]

5.2. City Development Planning and Existing Facilities

Prior to start to plan and project the sewer system of a city or a place, the city planning maps and 1/1000 scaled city development maps should be completed and ready. The projects, designed without these maps, are based on predictions. The planning maps and scaled city development maps should also be updated with the design projects during the preparation and application phases of the new projects due to rapid development of cities by the time. [7]

There should probably be adequate/inadequate existing sewer, septic tanks or drainage lines in the city. Before any new planning, the conditions of these existing facilities are identified and necessary measurements are taken in order to determine the sufficiency and abandonment. The ones which seem to be useable again, are repaired and became adequate. [7]

During the preparation of sewer system project before the placement of lines, the layout plans of existing water lines, electric lines, communication lines, gas lines and the levels of foundations with type of soils have to be known. If they are known; the related on-place survey measurements should be done. Also, the groundwater level and the fluctuations in the regions which have high groundwater level should be known. [7]

An essential care should be provided during the preparation of sewer system projects which has the objective of giving no damage to existing and adjacent structures or facilities. The necessary details of existing and future planned facilities must be included in the new plan to prevent any unexpected errors. The other facility operators should also own these maps in order to construct and repair the finished projects. The necessary permissions have to be taken from related establishments during construction and/or repair works to prevent the occurrence of any ordinary work. Briefly, nobody should break the others made.

5.3. Major Parameters of the Sewer System

The design of sewer system changes due to many factors of host country. The governmental regulations, construction techniques, materials available, topographic restrictions, economic conditions and etc. are all effective on the system performance. When it is considered as a single and typical type, the major parameters of this system are: population, water demand, sewer discharge, pipe diameter, line slope, and flow velocity.

Despite the similarity of equations and acceptances in hydraulic design calculations in many projects; the results may vary due to experiences and initiations. Sometimes costs get the priority, sometimes quality and time get the priority. The best selection is generally the presence of combinations. The optimization takes and important role during these circumstances to have the most effective solutions.

5.4. Population

Domestic wastewater quantities normally are to be computed on a contributing population basis. The population to be used in design depends upon the type of area which the sewer serves. If the area is strictly residential, the design population is based on full occupancy of all housing and quarters served. If the area served is entirely industrial, the design population is the greatest number, employed in the area at any time, even though some of these persons may also be included in the design of sewers in the residential area. For sewers serving both residential and industrial areas, the design population includes residents and nonresidents, but obviously no person should be counted more than once. [1]

The population values and the results of population estimation methods may show a little difference due to acceptances and techniques used. Naturally, a perfect certain result can not be taken by calculations because all methods are the outcomes of a prediction. The reliability of this prediction depends upon the accuracy of the method selected. Migrations, births, dead, developments and political approaches all play an important role in prediction. The population estimations in *SewDes 1.0* are made according to Specification of The Bank of Provinces (Kanalizasyon İşlerinin

Planlaması ve Projelerinin Hazırlanmasına Ait Talimatname Fasıl: 1, umumi Hükümler ve Planlama).

5.5. Water Demand and Sewer Discharge

Any analysis of the future demand of a particular community should always begin by considering present use. To the extent possible, consumption should be broken down by classes of users (domestic, commercial, industrial, public), area of the city, economic level of the users, season of the year, etc. The rather common procedure of dividing total use by total population to derive a per capita consumption should be applied only with great care. Since: [5]

1. The entire population may not be served by the municipal system
2. There may be large industrial uses which will not change with population
3. The characteristics as well as the size of the population may be changing.

Sanitary sewage wastes are derived principally from the water supply. Very little water is actually "consumed" in the sense that it is permanently removed from the community's environment. For this reason, estimation sewage flows should be prefaced by a study of both present water consumption and that expected in the future. The proportion of the water supplied which will reach the sewers depends in large part upon local conditions. In individual communities the sanitary sewage flow may vary from 70 to 130 percent of the water consumed. It is fairly common to assume that the average rate of sewage flow is equal to the average rate of water consumption, but this should be done only after careful consideration of the actual nature of the community. [5]

Sewage flow, like water consumption, will vary with the time of day, day of the week, season of the year, and weather conditions. The variations from the mean are less than those observed in water supply because the sewers do not flow full and thus provide a degree of equalization. Data have been collected in many communities which permit estimation of the ratio of peak and minimum flows to the average as a function of either average flow or population. [5]

Sanitary sewers carry domestic sewage, industrial waste, and whatever ground, surface, and storm water enters through joints, manhole covers, and defects in the system. Extraneous flows from groundwater infiltration enter the sewer system trough defective pipe, joints, fitting, manhole walls and submerged manhole covers. Infiltration is the water which enters sewers trough poor joints, cracked pipes, and the walls of manholes. Inflow enters through perforated manhole covers, roof drains connected to sewers, and drains from flooded cellars. [1, 5]

The amount of infiltration depends upon the care with which the sewer system is constructed, the height of groundwater table, and the character of the soil. An expansive soil will tend to pull joints apart and permit more leakage, while granular soils permit easy travel of water to the sewer where it may enter through joints or breaks. Since construction conditions and soil characteristics vary widely, infiltration is difficult to predict without actual flow measurements. Sewer size has little effect since, although large sewers have greater joint length, the workmanship is likely to be better than in small sewers. [5]

As a result the infiltration quantity can not be predicted exactly but have to be considered in the sanitary sewer design despite its being a little amount. Since sewers deteriorate with age, estimate of infiltration even for new system should be reasonably generous. An example for this infiltration flow can be given from Feasibility Research Study of Tempo Construction Company which has prepared the project study for The Bank of Provinces. The study uses infiltration flow as 0.1 lt/sn.ha. and mentions that this value should be reconsidered after a geotechnical survey.

In *SewDes 1.0* the infiltration of precipitation and contributing of groundwater to the system is considered as additional discharge and the addition of these discharges to residential discharge are provided by an input box for each line segment. By the way, the *sewer design discharge* is calculated.

5.6. Pipe Diameter and Type of Pipe

Pipe diameter selection for the design of sewer system is one of the most important sections. In case of having large market availability, it is very easy to arrange the exact dimension with respect to the design requirements. Market of pipes varies according to types and quality.

In hydraulic designs, the pipe diameters are mentioned as the inner diameters. The outer diameters are determined by adding the pipe thicknesses. In design phase the diameter can be selected for trial and can be changed with respect to the pipe market list. The first full flow diameter value can also be determined directly by using the discharge and velocity as performed in *SewDes 1.0*. Where:

$$Q = A \times V \tag{5.1}$$

$$Q = \left(\pi \times \frac{D^2}{4} \right) \times V \tag{5.1a}$$

here;

Q : The sewer discharge at flowing full (m^3/s)

A : Cross section area of pipe (m^2)

D : Pipe Diameter (m)

V : Velocity at flowing full (m/s)

The wetted perimeter is calculated by diameter. In the case the system design is performed according the full flow: the wetter perimeter can be found directly from the whole perimeter of pipe section. Where:

$$P = \pi \times D \tag{5.2}$$

here;

P : The wetter perimeter at full flow (m)

A : Pipe Diameter (m)

The equation of *hydraulic radius* for full flow is then as follows:

$$R = \frac{A}{P} \tag{5.3}$$

$$R = \frac{\pi \times \frac{D^2}{4}}{\pi \times D} \tag{5.3a}$$

$$R = \frac{D}{4} \tag{5.3b}$$

here;

R : Hydraulic radius

A : Pipe Diameter (m)

Pipe diameter affects the cost of project directly so the selection is very important. Selection of large pipe diameters are beside smaller diameters without any claim may cause an enormous cost increase in project prize estimation. Hence; the diameter has to be selected not only according to hydraulic requirements but also according to the project budget and engineering ethics.

The advantages and the disadvantages of larger and smaller pipe diameters affect the cost of project not only directly but also indirectly. The increasing pipe diameters affect the installation depth and so increase the excavation volume which means more money and time. The selection of the diameter generally depends on the design engineer who has to consider the hydraulic design requirements, installation phase, future use and maintenance phases, all of which includes the economic comparisons. The advantages of using small diameter pipes can be explained briefly as follows:

- Material costs less than larger diameter pipes
- Installation workmanship is easier and cheaper than larger ones
- Better joint sealing at connections
- Usually less excavation
- Usually needs steeper slope which provides high flow velocity thus causes low deposition in pipes
- Availability at many markets

The disadvantages of using small diameter pipes can be explained briefly as follows:

- Frequently plugging due to unexpected high flows or waste substances
- Have to be changed due to increasing population, have less flow tolerances

- Cause much maintenance cost and time due to frequent renewing.
- Needs more repair thus cause indispose in traffic and citizen's health.

Most citizens describe sewer system construction works as a real nightmare. This is because of the accompanied disturbances caused during the construction period, which nearly paralyzes the routine of the daily life. [11]

As explained before the pipe diameters have a great availability range in different types. In our country generally concrete and PVC type pipes are used. Using concrete type pipes has the advantage of casting in-place whenever larger diameters are needed. Some of the available pipe diameters in our country are listed in the table below:

Table 5.1 Some of the available concrete pipe diameters in different markets

Pipe Diameters Used by The Bank of Provinces	Pipe Diameters Used by The Ministry of Public Works Establishment	Pipe Diameters Used by A Traditional Production Company in Adana/Turkey
150 mm	200 mm	200 mm
200 mm	250 mm	300 mm
300 mm	300 mm	400 mm
400 mm	350 mm	500 mm
500 mm	400 mm	600 mm
600 mm	500 mm	700 mm
800 mm	600 mm	800 mm
1000 mm	800 mm	900 mm
1200 mm		1000 mm – 2200 mm

The selection of pipe diameter during the hydraulic calculations is made according to the next bigger one available in the market. The example explains this better: let hydraulic calculation determined the pipe diameter as 370 mm, than the selection will be 400 mm. Let hydraulic calculation determined the pipe diameter as 180 mm, than the selection will be 200 mm. In *SewDes 1.0* this selection is made automatically according to the pipes available in the market list.

The diameter selection sometimes has a confusing such as diameters found both suitable for upper next or previous one in the market. Selecting the pipe always as larger one is not an economic solution, for example the diameter found as 216 mm in the market list of The Bank of Provinces seems to be taken as 300 mm however *SewDes 1.0* selects 200 mm for this circumstance because of a *pipe choose ratio* which is taken as 0.25 and can be changed easily by the designer. The *pipe chooser ratio* works between the interval of two successive diameters and divides the interval to quarters then selects the diameter whether it is in the first quarter which is nearer to smaller diameter, or selects the diameter whether it is in the other ¾ quarter section which is nearer to larger one.

Beside the pipe diameter, the type of pipe selected is also very important. The type of pipe designates the *roughness coefficient* which refers the *manning coefficient* in design. The manning coefficient is showed with letter "n". This coefficient is input by the designer in *SewDes 1.0* at inputs and constraints part.

Values of n to be used in the design generally range from 0.013 to 0.015. The lowest n values apply to new or relatively new pipe (in sections greater than 150 mm) with smooth interior surfaces, smooth bore, even joints, in excellent to good condition and well constructed. Higher n values are required for older pipe with rough interior surfaces, open or protruding joints, in fair to bed condition and poorly constructed. Values up to 0.017 are often justified for very old pipe (such as brick or block sewers) in extreme deterioration, or pipe very poorly constructed with improper alignment, sags and bellies, cracked or offset joints, broken wall sections or internal corrosion. Some manufacturers of plastic and asbestos cement pipe report n values of 0.009 to 0.011. However, due to uncertainties in design and construction, plus a desire to provide a margin of safety, n values smaller than 0.013 will not normally be permitted. Variation of n with depth of flow can be shown experimentally, and may be considered in designing sewers to flow partially full. [1]

For circular conduits, Camp [13, 14] was able to show that the *n* value for a conduit flowing partially full is greater than that for a full conduit. Using measurements on clean sewer pipe and drain tile, both clay and concrete from 4 to 12 in. in size, he found an increase of about 24% in the n value at the half-depth. The n value for the pipe flowing full was found to vary from 0.0095 to 0.011. Taking an average value of 0.0103, the n value at half-depth should be about 0.013. This is identical with the usual design value, which is based largely on measured values in sewer flowing partially full. [12-14]

The value of n is affected by surface roughness, vegetation, and, its seasonal change, channel irregularity, channel alignment, silting and scouring, suspended material and bed load, obstruction, stage and discharge etc. The Table 5.2 gives a list of n values for various kinds (Closed conduits flowing partly full). For each kind the minimum, normal and maximum values of n are shown. For the case in which poor maintenance is expected in the future, values should be increased according to the situation expected. This table is very useful as a guide to the quick selection of the n value to be used in problems. [2, 12]

Sewers are usually made plain concrete, reinforced concrete, asbestos cement, cast iron or corrugated steel. The selection of a specific type of sewer material is governed by the quantity of sewage and the stresses applied. Minimum proposed pipe diameters are Ø200 mm for sanitary sewers and Ø150 mm for house connections. [2]

The roughness coefficient value is preferred as 0.015 for sewer concrete pipes. In design of sewer systems, especially the pipe diameters available in the markets should be used for calculations. The diameters less than 200 mm should not be used for sanitary manhole connections except house connections. [3]

Table 5.2 Values of the roughness coefficient, *n*

A- METAL	**Minimum**	**Normal**	**Maximum**
Brass, smooth	0.009	0.010	0.013
Lock bar and welded steel	0.010	0.012	0.014
Riveted and spiral steel	0.013	0.016	0.017
Coated cast iron	0.010	0.013	0.014
Uncoated cast iron	0.011	0.014	0.016
Black wrought iron	0.012	0.014	0.015
Galvanized wrought iron	0.013	0.016	0.017
Corrugated metal (Subdrain)	0.017	0.019	0.021
Corrugated metal (Storm drain)	0.021	0.024	0.030
B-NONMETAL			
Lucite	0.008	0.009	0.010
Glass	0.009	0.010	0.013
Cement (Neat surface)	0.010	0.011	0.013
Mortar	0.011	0.013	0.015
Concrete Culvert, straight and free of debris	0.010	0.011	0.013
Concrete Culvert, with bends & some debris	0.011	0.013	0.014
Finished concrete	0.011	0.012	0.014
Sewer with manhole, inlets, etc., straight	0.013	0.015	0.017
Unfinished concrete, steel form	0.012	0.013	0.014
Unfinished concrete, smooth wood form	0.012	0.014	0.016
Unfinished concrete, rough wood form	0.015	0.017	0.020
Wood (Stave)	0.010	0.012	0.014
Wood (Laminated, treated)	0.015	0.017	0.020
Clay (Common drainage tile)	0.011	0.013	0.017
Vitrified sewer	0.011	0.014	0.017
Vitrified sewer with manholes, inlet, etc.	0.013	0.015	0.017
Brickwork (Lined with cement mortar)	0.012	0.015	0.017
Sanitary sewers coated with sewage slimes, with bends and connections	*0.012*	*0.013*	*0.016*
Rubble masonry, cemented	0.018	0.025	0.030

5.7. Ground Slope and Line Slope

Line slope is another main parameter for the design of sewer systems. In gravity sewers the aim is to convey the discharge with the same slope of ground if it is possible and suitable. The slope value is limited for design calculations where a minimum and a maximum limit values are available. Nearby these limit values; recommended slope values are also available with respect to pipe diameters.

Slope also affects the cost of project directly where it arranges the excavation amount and pipe diameter selection. In the case of reducing the amount of excavation it seems to reduce the slope value at first. The approach is logical but the results are not correct all the time. Slope is a kind of key variable, where it plays an important role not only in the hydraulic calculations but also in cost estimation investigations.

In the typical design application, with a known flow, a desired minimum velocity and terrain restrictions on slope presented by terrain and subsurface conditions, there will usually be a number of possible combinations of pipe size and slope which will serve. *A small pipe on a steeper slope can carry the same flow as a larger line on a flat slope. Which is the best choice is not always clear, although as a rule it is cheaper to use a larger pipe if this reduces the excavation required. This may not always occur, however, since the drop in elevation of the invert occasioned by the change in size may be greater than the difference in drop between the two slopes*. [5]

Assuming uniform flow, the values of slope in the Manning formula is equivalent to the sewer invert slope. Pipe slopes must be sufficient to provide the required minimum velocities and depths of cover on the pipe. Although it is desirable to install large trunk and interceptor sewers on flat slopes to reduce excavation and construction costs, the resulting low velocities may deposit objectionable solids in the pipe creating a buildup of hydrogen sulfide, and thus will be avoided. [1]

During the design calculations of sewers the ground elevations are taken in consideration and thus the street slopes (ground slope) and crown elevations of upper manholes become very important. In this circumstances we generally meet with three conditions below: [3, 7]

1. *The ground slope is less than the minimum line slope ($S_g < S_{min}$)*

In this case at the beginning the upper crown is selected according to the requirements of minimum depth cover and the slope is selected as the minimum line slope. If the channel depth is more than the permitted maximum depth then; a pump should be used to carry the flow to the minimum depth cover level which refers the calculated crown elevation of the lower manhole.

2. *The ground slope is bigger than the maximum line slope ($S_g > S_{max}$)*

Here, the pipes are laid down with the maximum line slope. In this case drop manholes are constructed at the points where the heights are equal to minimum height (minimum cover depth). Drops can be performed at the upper manholes which are called *drop manholes.* The drop values can be found by simple geometric methods and also can be calculated by the equation below:

$$Drop = \left[E_{uc} - \left(S_l \times L \right) \right] - (E_{gl} - h_{\min}) \tag{5.4}$$

here;

E_{uc} : Upper manhole crown elevation

S_l : Line slope (sewer invert slope)

L : The distance between two manholes (upper and the lower) (line length)

E_{gl} : Lower manhole ground elevation

h_{min} : Minimum depth cover

3. *The ground slope is between max. and min. line slope ($S_{min} < S_g < S_{max}$)*

At the beginning the upper crown is calculated according to minimum depth cover and the slope is selected as the same of ground slope. The most applicable one is this case. In this case some special conditions occur such as:

a) If the depth cover of the upper manhole is bigger than the required minimum depth cover, then the line length is taken in consideration for solution.
    - The arrangements will be performed as: the depth cover will be minimum value at the lower manhole and a slope will be selected bigger or equal to minimum line slope.

- If the length of the street is long then the minimum slope is selected as line slope and it is used till where it reaches to a minimum depth cover value. Thereafter the line slope is selected as ground slope.

b) If the line is to be connected to an another line at downstream
   - If the other line is more steeper: a slope is selected as it is going to be connected to this line or a more flat slope is used till to connection point and a drop is to be made at connection.
   - If there will be additional flow from other lines, then the street ending elevations of the lateral connecting lines get an extra importance. In this case the crown elevation of lower manhole is calculated in such a way that the line can able to take the other additional flows and by the way an adequate slope is selected with respect to this situation.

In *SewDes 1.0* the slope is determined according to the cases above regarding the aim of this research study and the trial of slopes for adequate velocities and cover depths are satisfied by increasing the minimum slope values by an addition of 0.0005. The slope increase is limited at maximum slope and the adequate slope is written in the table at the end of computation. Maximum and minimum slopes are calculated by using Manning's equation and the ground slope is calculated with respect to upper and lower manhole ground elevations dividing by line length. Where:

$$S_g = \frac{E_{ug} - E_{lg}}{L} \tag{5.5}$$

here;

S_g : Ground slope

E_{ug} : Ground elevation of upper manhole

E_{lg} : Ground elevation of lower manhole

L : Length of line between upper and lower manhole

A diameter is calculated by using equation (5.1) and this is used in Manning formula to obtain minimum and maximum slopes. Where:

$$V = \frac{1}{n} \times R^{2/3} \times S^{1/2} \qquad (\textit{Manning formula}) \qquad (5.6)$$

$$R = \frac{r}{2} \qquad (\textit{For full flowing pipes}) \qquad (5.6a)$$

$$S_m = \left(\frac{V_m \times n}{\left(\frac{r}{2}\right)^{2/3}} \right)^2 \qquad (\textit{Slope formula}) \qquad (5.6b)$$

here;

V_m : Minimum and maximum velocities in sequence

S_m : Minimum and maximum slopes in sequence

n : Manning coefficient

R : Hydraulic radius

r : Pipe radius

The slopes are expressed as 1/A in specification of The Bank of Provinces. The slope has to be determined in order to provide a flow velocity of 0.5 m/s and a water height of at least 2 cm. Cleaning systems should be made where the minimum requirements of depth cover and velocity are not satisfied. [8]

There are many practical methods to determine slope values without calculation but using experience, slope tables or existing projects. In the case of its importance on the formation of cost, it has to be determined carefully and adequate values should be selected. An optimization is always recommended for this circumstance which is easy to try a few different values and to find out possible combinations. Nowadays computer programs are very improved and many calculations can be resulted in a few seconds which helps engineers in designing projects having many variables.

The table given below can just be used as a guide in the design works. The slope values can be only used by considering the velocity and water height restrictions. [8]

Table 5.3 Slope table of The Bank of Provinces (Specification 1991)

Lines	*Minimum slope*	*Normal slope*	*Exceptional slope*	*The most adequate slope*
Ø2150 mm	1:100	1:15	1:7	1:50
Ø200-Ø300 mm	1:300	1:15	1:7	1:50 - 1:150
Ø350-Ø600 mm	1:500	1:25	1:15	1:100 - 1:200
Ø650-Ø1000 mm	1:1000	1:50	-	1:200 - 1:500
Ø1000-Ø2000 mm	1:3000	1:75	-	1:300 - 1:750

5.8. Flow Velocity

Velocities in sewers are selected with the goal of keeping the solids in the sewage in suspension or at least in traction. Sanitary sewers should be sized to provide a velocity of at least 0.6 m/s (2ft/s), which is adequate to keep grit in traction. Some regulatory agencies specify minimum slopes for sewers of various diameters. These slopes are those which are calculated to give a velocity of 0.6 m/s when the sewers are full. Since the sewers are commonly not full and the hydraulic radius is thus different from that of a full sewer, the actual velocity will differ from 0.6 m/s, generally being less. In flat terrain the designer may be tempted to use larger pipes since the “minimum” slope is less. This is not good practice, since a larger sewer carrying a low flow will have a velocity far less than that corresponding to full flow.[5]

To prevent any settlement in the pipe it is aimed to keep the velocity over 0.6 m/s. Maximum velocity is determined as 2.5 - 3.0 m/s by The Bank of Provinces. High velocities cause not only corrosion and destruction in the pipe but also form forces where the flow directions change. Thus the high velocity values are not preferred. [3]

Velocities are calculated by using Manning formula where it is expressed in equation 5.6. The velocity is expressed as capital letter “V” when it is full flow and small letter “v” during actual flow. In *SewDes 1.0* the “v” refers to the design velocity and minimum velocity is taken as target value during design.

5.9. Inputs and Constraint

As seen in many design studies there always some constraints exist on variables with respect to local regulations, technical restrictions or some present mandatory acceptances. These constraints are generally used to provide the most adequate solutions in logical limits of variable values. Specifications, manuals, guides, handbooks, examples and so many assistant aided programs are presented for designers and/or manufacturers.

In sewer system designs some restrictions also exist for inputs and thus outputs. The inputs of design phase generally show similarity despite the variety of projects. The inputs used in sewer design are restricted mainly by hydraulic reasons and operational reasons which are designated by scientific experimental results and engineering logics. Naturally, the constraints on inputs form limited values for outputs, which ease the control of computation results whether they are logical or not. Although every input value has its own constraint; it is not always taken in consideration like the being of a day 24 hours. Some common values are used as they are accepted due to scientific proofs. As a very simple example: if a time interval for "return period" of water will be used as an input value for design calculation it is unnecessary to clarify that it can not be more than 24 hours.

Constraints on variables for sewer system design according to local obligations of governmental establishments (The Bank of Provinces is taken in consideration here) and scientific findings are as follows:

Project life is taken as 30 years and 5 years is accepted as construction period. The reports are prepared by considering the population for the next 30 years period.

Population increase coefficient has the value between 1 and 3 according to population forecasting method which is used in specification.

Water demand accepted for the population is taken from the related tables (4.3) where the unwritten values can be obtained by interpolation method.

Maximizing coefficient of water demand used as 1, 5 due to summer season. This coefficient is multiplied by the average daily water demand.

Return period which refers the return time of consumed water to sewer is taken as 12 hours. Thus the P2 value changes with respect to this value.

Return percentage which refers to amount of consumed water returning to sewer is taken as %100.

Minimum velocity which refers to minimum full flow velocity for self-cleansing is taken as 0, 5 - 0, 6 m/s and *Maximum velocity* is taken as 2, 5 - 3, 0 m/s.

Fullness ratio is taken at least %10 and at most %80 for sewer systems. Pipes are accepted to be full during design. Maximum %60 ratio is recommended.

Manning coefficient is used both 0.013 and 0.015 but 0.015 is preferred where the pipe inner surfaces are not staying same as they are new. See Table 5.2

Maximum manhole interval is generated from pipe diameters. See Table 5.4

Table 5.4 Maximum manhole intervals

Pipe Diameter (cm)	*Maximum Manhole Interval (m)*
20 – 55	50
60 – 80	70
90 – 140	90
Larger than 140	125 – 150

Minimum cover depth refers to the height of adequate protective soil cover over pipe. The minimum value is taken as 1.50 – 1.70 m.

Maximum excavation depth refers to the maximum height of trench excavation starting from ground level and ending at the bottom of trench which is max. 4 m.

Minimum and maximum slope values are given in Table 5.3. Minimum slope is calculated with respect to minimum velocity.

Drops which are at most 0.40 m, they are formed inside manholes and the ones up to 2.0 m are formed with special steel pipes inside manhole.

In *SewDes 1.0* the constraints are asked at the beginning of design to designer and hydraulic calculations are done according to those values where the example is presented in other next section.

5.10. Design Procedure and Hydraulic Calculations

After a preliminary layout has been made, tabulation will be prepared in convenient form setting forth the following information for each sewer section: [1]

- Designation of manholes by numerals
- Contributing populations – resident and nonresident
- Design flows
- Length of sewer
- Invert elevations
- Line slope
- Pipe diameter and roughness coefficient
- Floe depths at design flows
- Velocities at design flows.
- Depths of cover on the pipe

Generally, it is not desirable to design sewers for full flow, even at peak rates. Flows above 90 to 95 percent of full depth are considered unstable, and may result in a sudden loss of carrying capacity with surcharging at manholes. In addition, large trunk and interceptor sewers laid on flat slopes are less subject to wide fluctuations in flow, and if designed to flow full may lack sufficient air space above the liquid to assure proper ventilation. Adequate sewer ventilation is a desirable method of preventing the accumulation of explosive, corrosive or odorous gases and of reducing the generation of hydrogen sulfide. Therefore, trunk and interceptor sewers will be designed to flow at depths not exceeding 90 percent of full depth; laterals and main sewers, 80 percent; and building connections, 70 percent. However, regardless of flow and depth the minimum sizes to be used are 150 mm for building connections and 200 mm for all other sewers. [1]

The flow in sewers is intended to be gravitational. However, pumping facilities may also be provided if topographic conditions are not favorable for a gravity system. Sewers are designed as open channels, flowing partly full or, at most just full. Because of hydraulic requirements, sewers are designed with circular cross-sections. Geometric elements of a sewer are shown in Figure 5.1. In the hydraulic computations, the percent fullness of a sewer is defined by the depth ratio d/D. [2]

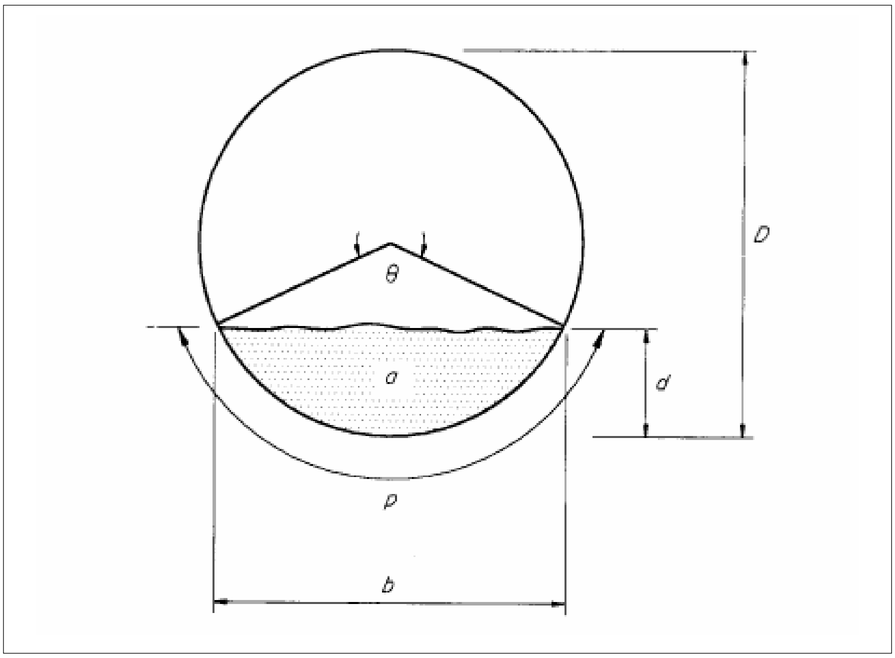


Figure 5.1.Definition of parameters for open channel flow in a circular sewer.

The flow in simplified sewers is always open channel flow – that is to say, there is always some free space above the flow of wastewater in the sewer. The hydraulic design of simplified sewers requires knowledge of the area of flow and the hydraulic radius. Both these parameters vary with the depth of flow, as shown in Figure 5.1. From this figure, trigonometric relationships can be derived for the following parameters:

(1) The **area of flow** *(*a), expressed in m^2;

(2) The **wetted perimeter** *(*p), m;

(3) The **hydraulic radius** (r), m; and

(4) The **breadth of flow** *(*b), m.

The hydraulic radius (sometimes called the hydraulic mean depth) is the area of flow divided by the wetted perimeter. The breadth of flow is used for the calculation of the risk of hydrogen sulphide.

Parameters 1 – 4 above depend on the following three parameters:

(5) The **angle of flow** (θ), expressed in *radian*s;

(6) The **depth of flow** *(*d), m; and

(7) The **sewer diameter** *(*D), m.

If the angle of flow is measured in degrees, then it must be converted to radians by multiplying by (2π/360), since 360^{o} equals 2π radians.

The ratio d/*D* is termed the **proportional depth of flow** (which is dimensionless). In simplified sewerage the usual limits for d/*D* are as follows:

$$0.2 < d/D < 0.8$$

The lower limit ensures that there is sufficient velocity of flow to prevent solids deposition in the initial part of the design period, and the upper limit provides for sufficient ventilation at the end of the design period.

The equations are as follows:

(a) *Angle of flow:*

$$\theta = 2\cos^{-1}\left[1 - 2\left(\frac{d}{D}\right)\right] \tag{5.7}$$

(b) *Area of flow:*

$$a = \frac{D^2}{8}(\theta - \sin\theta) \tag{5.8}$$

(c) *Wetted perimeter:*

$$p = \frac{\theta \times D}{2} \tag{5.9}$$

(d) *Hydraulic radius (= a/p):*

$$r = \left(\frac{D}{4}\right) \times \left(1 - \frac{\sin\theta}{\theta}\right) \tag{5.10}$$

(e) *Breadth of flow:*

$$b = D \times \left(\sin\frac{\theta}{2}\right) \tag{5.11}$$

When $d = D$ (that is, when the sewer is considered to flow full), then;

$$a = A = \pi \times \left(\frac{D^2}{4}\right)$$

$$p = P = \pi \times D$$

$$r = R = \frac{D}{4}$$

The following equations for (*a)* and (*r)* are used in designing simplified sewers:

$$a = k_a \times D^2 \tag{5.12}$$

$$r = k_r \times D \tag{5.13}$$

The coefficients (k_a) and (k_r) are given from equations 5.8 and 5.9 as:

$$k_a = \frac{1}{8} \times (\theta - \sin\theta) \tag{5.14}$$

$$k_r = \frac{1}{4} \times \left(1 - \frac{\sin\theta}{\theta}\right) \tag{5.15}$$

When $a = A$ and $r = R$, then $k_a = p/4$ and $k_r = 0.25$.

Where **θ** is the central angle, **d** is the depth of flow, **D** is the diameter of sewer, **a** is the flow area, **p** is the wetted perimeter and **r** is the hydraulic radius. In the hydraulic computations, capital letters are used to characterize full flow whereas small-case letters are used for partially-filled flow conditions. The average cross sectional velocity, ***u***, in terms of m/s can be determined from Manning's equation: [2]

$$u = \frac{1}{n} \times r^{2/3} \times \sqrt{s_f} \tag{5.16}$$

Where n is the Manning roughness coefficient which can be obtained from Table 5.2 and s_f is the friction slope in case of partially filled sewers. For large projects composed of thousands of sewers, determination of geometric elements for partially filled sewers is not practical. Therefore, it is convenient to express the hydraulic quantities of circular sewers in charts which are in dimensionless forms. The dimensionless velocity, u/U and discharge q/Q are obtained as follows: [2]

$$\frac{u}{U} = \left(\frac{N}{n}\right) \times \left(\frac{r}{R}\right)^{2/3} \times \left(\frac{s_f}{S_f}\right)^{1/2} \tag{5.17}$$

$$\frac{q}{Q} = \left(\frac{a}{A}\right) \times \left(\frac{N}{n}\right) \times \left(\frac{r}{R}\right)^{2/3} \times \left(\frac{s_f}{S_f}\right)^{1/2} \tag{5.18}$$

Domestic sewage contains some solid wastes from bathrooms, toilets, laundries and kitchens whereas storm water contains some sand, gravel and debris of many origins. When the flow velocity in sewers is low, the suspended material may settle down, which decreases the hydraulic efficiency. Therefore, a minimum allowable velocity, u_{min}, tractive force, $\gamma r s_f$, and slope, S_c, are to be maintained in order to eliminate the accumulation of the material. Equal self-cleansing properties at all depths can be achieved if the tractive forces are the same in case of full and partially-filled sewers:

$$\gamma R S_f = \gamma r s_f$$

From which one obtains $s_f = (R/r) S_f$. By inserting the value of s_f into the dimensionless velocity and discharge (Equation 5.17 and 5.18 respectively), dimensionless self cleansing velocity u_s/U and discharge Q_s/Q, can then be obtained as: [2]

$$\frac{u_s}{U} = \left(\frac{N}{n}\right) \times \left(\frac{r}{R}\right)^{1/6} \tag{5.19}$$

$$\frac{q_s}{Q} = \left(\frac{N}{n}\right) \times \left(\frac{a}{A}\right)\left(\frac{r}{R}\right)^{1/6} \tag{5.20}$$

Hydraulic elements of circular sewers with equal self-cleansing properties at all depths are given in Figure 5.2.

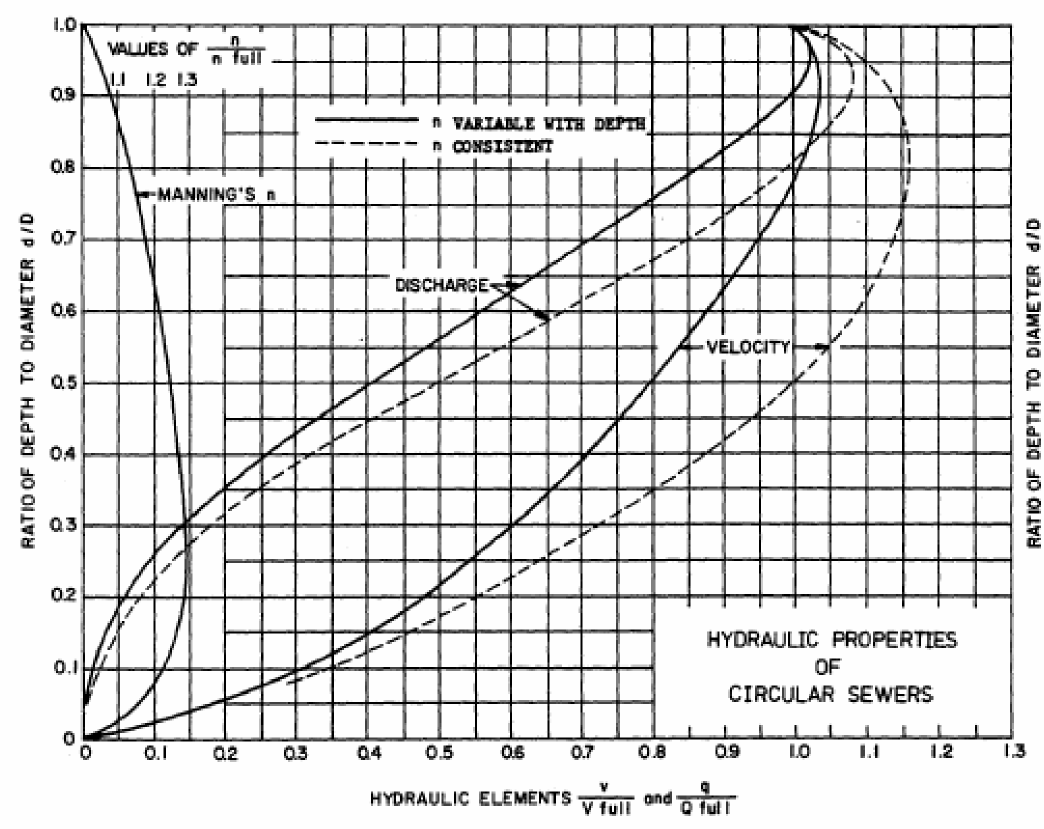


Figure5.2. Dimensionless hydraulic elements of circular sewers

The full flow capacity of circular pipes may be calculated directly from Manning's equation. The monograms permit a graphical solution of this problem, which is sometimes quicker than computational techniques if only a few conduits are involved. [5]

5.10.1. Hydraulic design by SewDes 1.0

In *SewDes 1.0* monogram is not used, all hydraulic calculations are made through equations. The hydraulic design calculation is summarized by steps respectively as:

1. Design flow is which the total of residential discharge and additional discharge is converted to full flow discharge (due to design for full flow) by using fullness ratio and Figure 5.2. where;

   $FR = \frac{q_d}{Q}$ => $Q = \frac{q_d}{FR}$;here Q is converted to m^3/s

2. Ground slope is calculated by using the equation (5.5).

3. A diameter value is calculated with respect to minimum velocity using the equation (5.1a)

4. The diameter is put into Manning equation (5.6) where the velocity is taken as minimum velocity value. In this way, the minimum slope value is obtained. The Manning coefficient is used as the one input manually.

5. The diameter is put into Manning equation (5.6) where the velocity is taken as maximum velocity value. In this way, the maximum slope value is obtained. The Manning coefficient is used as the one input manually.

6. Now, the three slope values (S_{min}, S_{max} and S_g) are compared.
   - If ground slope is less than maximum slope and than bigger than minimum slope then the ground slope is taken as line slope.
   - If ground slope is less than minimum slope then the minimum slope is taken as line slope.
   - If ground slope is bigger than maximum slope then the maximum slope is taken as line slope.

   The selection of line slope is not depending only on maximum and minimum values but also the cover depth is affecting this selection. The conditions written above are applied when the upper manhole crown depth is equal to the minimum cover depth.

As a condition when the upper manhole crown depth is bigger than minimum cover depth and when the ground slope is bigger than the calculated maximum slope, then the crown depth of lower manhole is checked with respect to minimum cover depth and if it does not satisfy the requirement, the selected line slope value is increased by 0.0005 for making a trial to satisfy the minimum requirements for cover depth.

When the ground slope is bigger than the maximum slope with a minimum cover depth value at the upper manhole, in this case a drop is unavoidable where it is calculated by equation (5.4).

Generally the crown elevations of upper manholes are selected as the minimum values but sometimes the conditions may be compelling. In this case the alternative solutions are applied as mentioned above.

7. By putting the equation (5.6) into (5.1) and solving them together, a diameter value is obtained for full flow. This diameter value may not be similar with the diameter values in markets. This computed value is converted to the values available in the market list of *SewDes 1.0* where the designer has the chance of forming a market list for each project. As an example when the diameter is calculated as 195 mm by the program then it is taken as 200 mm which should be the first minimum diameter available in the market list.

8. Using this diameter value in Manning equation (5.6) again, a full flow velocity is calculated.

9. The full flow velocity calculated in step 8, is converted to design velocity by using the v/V value obtained from Figure (5.2). The multiplication of full flow velocity (V) with the value obtained from related figure gives the design velocity.

10. The design velocity is compared with the minimum velocity and an engineering decision is made there. The diameter can be changed to make new trials or slope can be changed to make new trials. This case is very

important due to formation of cost values, so an optimization is made by *SewDes 1.0* to present to alternative solutions regarding cost.

In the first design stage of *SewDes 1.0* the diameter is considered for optimization and the design velocity is computed in accordance with the minimum requirements and available pipe diameters in market list. The design velocity is tried to be bigger than the minimum velocity value by changing the pipe diameters where the line slope is kept constant. By this way, the most adequate pipe with a minimum slope is being arranged and tabulated in the first table.

11. The second case in which the diameter value is kept constant and the line slope value is changing, is performed when the *optimization* button is clicked. Here, the slope is being increased by 0.0005 adding on the initial line slope where the diameter is kept constant. This trial is just accomplished for the diameter values that were formed during the first case. This method is applied in order to compare the results in the same frame and under same conditions.

12. After calculating the diameter and slope values, the crown and invert elevations are calculated by means of slope and diameter. Where:

$$E_{lc} = E_{uc} - (L \times S_l) \tag{5.21}$$

$$E_{ui} = E_{uc} - D \tag{5.22}$$

$$E_{li} = E_{lc} - D \tag{5.23}$$

here;

E_{lc} : Lower crown elevation (m)
E_{uc} : Upper crown elevation (m)
S_l : Line slope
E_{li} : Lower invert elevation (m)
E_{ui} : Upper invert elevation (m)
L : Length of line between upper and lower manhole (m)
D : Pipe diameter (m)

13. The excavation is calculated by using the ground elevation, invert elevation, pipe thickness and sand bedding height. The section for excavation area may not be rectangular due to probable side slope. In this case the side slope is calculated by (1: n) value which is to be input by the user. If there is not any side slope then there is no need for user to click the related button. (Where 1 is vertical and n is horizontal). The bottom width of the channel (trench) is calculated in accordance with the specifications of The Bank of Provinces.

Where;

Space near pipe (b)	Pipe Diameter (D)
20 cm	Ø200 – Ø400
35 cm	Ø400 – Ø700
60 cm	D > Ø700

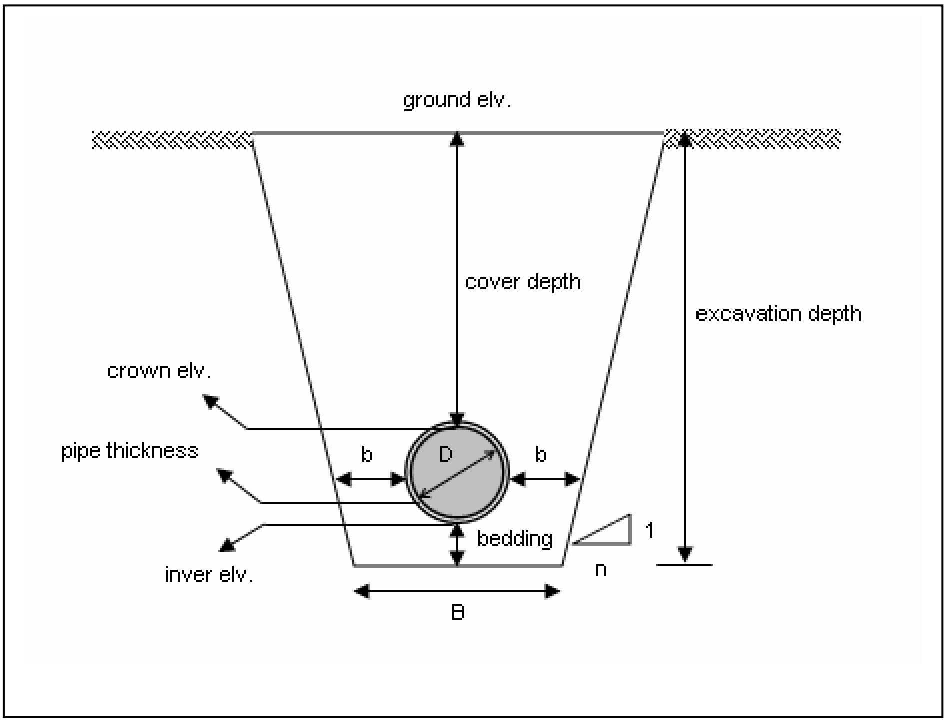


Figure5.3. Section of a typical sewer trench

The figure 5.3 illustrates the variables in a better way. The trench bottom width (B) is calculated as:

$$B=(2\times b)+(D_{outer}) \tag{5.24}$$

The height of sand bedding under pipe is calculated by the formula below:

$$h_{sb} = \frac{D}{10} + 10 \text{ cm} \tag{5.25}$$

here;

h_{sb} : Height of sand bedding under pipe (cm)

D : Outer diameter of pipe (cm)

14. The cost of pipe and installation and cost of excavation is calculated by using the unit prices which are input by the user. The government used unit prices are input to *SewDes 1.0* temporarily just to help to user. The pipe and installation cost is determined by multiplying the interval length by the related price, the excavation cost is determined by multiplying the excavation amount by unit price. This total amount is compared with the budget and the condition is written in *Remark* section. If the total amount at present is more than the budget then it is written as *Overload Budget.*

15. The maintenance cost is calculated throughout the experienced views of the civil engineers working in The Bank of Provinces in Adana. The maintenance length of pipe line is considered as 3 meters for one manhole interval. The repetition of maintenance in the project life time (*N*) is taken as a variable which is changing according to diameter of pipe. Where;

Pipe Diameter (mm)	Repetition for Every *N* Year
Ø200	3
Ø250	4
Ø300	5
Ø350	6
Ø400	6
Ø500	8
Ø600	10
Ø800	10
Ø1000	15
Ø1200	15

$$c_m = \left(\frac{C_p + C_e}{L} \right) \times 3 \tag{5.26}$$

$$C_m = c_m \times \left(\frac{Y_{pl}}{N} \right) \tag{5.27}$$

here;

c_m : Maintenance cost for one time (YTL)

C_p : Total cost of pipe material and installation (YTL)

C_e : Total cost of trench excavation (YTL)

L : Length of line between two successive manholes (m)

3 : Specified average length for maintenance (m)

C_m : Total maintenance cost in projected life year period (YTL)

Y_{pl} : Projected life year (number of years)

N : Repetition year of maintenance (Year)

16. The total cost is calculated by the addition of pipe cost, excavation cost and maintenance cost.

Despite this cost estimation does not show a complete reality, it just gives an idea to the designer to select the adequate pipe diameter and slope among several alternative combinations for a sewer line. The accuracy of cost estimation depends on many factors many of which are not considered in this study because of not being in the objective of this research. Cost estimation breakdown can be made throughout a wide analysis by considering every probable condition and equipment that will be included in the installation and operation of the system. It is certain that the sufficiency of hydraulic calculations is not enough to decide whether a system is adequate or not. Thus, a cost analysis should be made because sometimes the maintenance and operation costs cause a reverse effect on selected choices although they seem to be the best alternative.

The results of calculated equations are tabulated in *SewDes 1.0* in design phase. Similarly, a tabular result document exists in the specifications of The Bank of Provinces and other related establishments which ease the control of variables.

5.10.2. Optimization in SewDes 1.0

It is known that the engineering economics deals with the determination of minimum cost and maximum output for technical and business enterprise projects. The problem may neither be the solution of an existing matter nor the evaluation of a future aimed project; the objective is same: the selection of the most adequate solution among the all alternatives. [15]

…

The determination of the biggest and the smallest value refers to maximization and minimization. Constraints may always occur in projects. If it is projected to perform maximization or minimization in constrained projects then the reached solution is called *optimization*. Hence, optimization is the process of finding solution sets in order to reach to the most adequate result and to get it. [15]

Due to surplus of constraints in sewer design, it is naturally difficult to perform an optimization in mathematical equation form although it may be done for different researches. *In SewDes 1.0* the optimized results are determined by minimizing and maximizing operations regarding the constraints of designer. Briefly, the applied method can be summarized as follows:

Pipe Cost= (Length of Line) x (Pipe Unit Price)

$$C_p = L \times P_{pup} \tag{5.28}$$

Excavation Cost= (Line Length) x (Trench Section Area) x (Excavation Unit Price)

$$C_e = L \times A_{sa} \times P_{eup} \tag{5.29}$$

Maintenance Cost= {[(3) x (Excavation Section Area) x (Excavation Unit Price)] + (3) x (Pipe Unit Price)} x (Project Life Year / Repetition)

$$C_m = \left(\frac{Y_{pl}}{N}\right) \times \left[\left(3 \times A_{sc} \times P_{eup}\right) + \left(3 \times P_{pup}\right)\right] \tag{5.30}$$

Total cost is the addition of all three costs, where:

$$C_T = C_p + C_e + C_m \tag{5.31}$$

Minimization of total cost can be maintained by minimizing the each cost. Where;

$$\text{Min.}C_T = \text{Min.}C_p + \text{Min.}C_e + \text{Min.}C_m \tag{5.32}$$

$$\text{Min.}C_p = L \times P_{pup} \text{(having the minimum diameter)} \tag{5.33}$$

$$\text{Min.}C_e = L \times A_{sa}\text{(Mindiameter,Mincoverdepth,Minslope)} \times P_{eup} \tag{5.34}$$

$$\text{Min.}C_m = 3 \times \left(\frac{\text{Min.}C_p + \text{Min.}C_e}{L} \right) \tag{5.35}$$

As it is seen from the minimizing equations, the optimum results are formed by minimizing the variables which are pipe diameter, slope, and cover depth. The hydraulic calculations are performed regarding the minimum aspects of constraints hence; these parameters get the optimum values. This can be explained with an example of cover depth where it is always tried to use minimum cover depth value to minimize the excavation amount and excavation cost.

The parameters of the function f (Total Cost) are:
fC_T (diameter, slope, cover depth) then;
The parameters of the function min.f (Total Cost) are:
Min. fC_T ($D_{min,}$ $S_{optimum,}$ h_{min})

The maintenance cost which is calculated according to today conditions may be converted to its future worth with the compound interest formulas and flow diagrams, but then all of these future worth will be converted to present worth again to make a simple comparison with today's financial conditions. In this case, the conversion of present maintenance cost (which is calculated as *present* worth) to future worth and making the same reverse computation again is meaningless. So; the calculated cost value is used directly as present worth which is reflecting the future worth. This condition is tried to be explained in Figure 5.4.

Conversion of a present worth to future worth is made by using the compound interest formulas written in the table below:

Table 5.5 Compound interest formulas

Output	*Input*	*Formula*	*Symbol*
F (Future)	P (Present)	$(1+i)^n$	(F/P, % i, n)
P (Present)	F (Future)	$\frac{1}{(1+i)^n}$	(P/F, % i, n)

The converted future value will be re-converted to present value with the same interest rate and time. As seen from the table, the interest formulas are reverse of each other so the calculated maintenance cost can be used directly without any conversion. The repetition of maintenance cost in different years is shown in the cash flow diagram below where it is not required to convert them to each other again because in the present worth method, all the benefits and cost of the system throughout its life time are transferred to the beginning of the project and the alternative which gives the maximum net present worth is chosen as the best alternative. Here, the minimum present worth is taken in consideration.

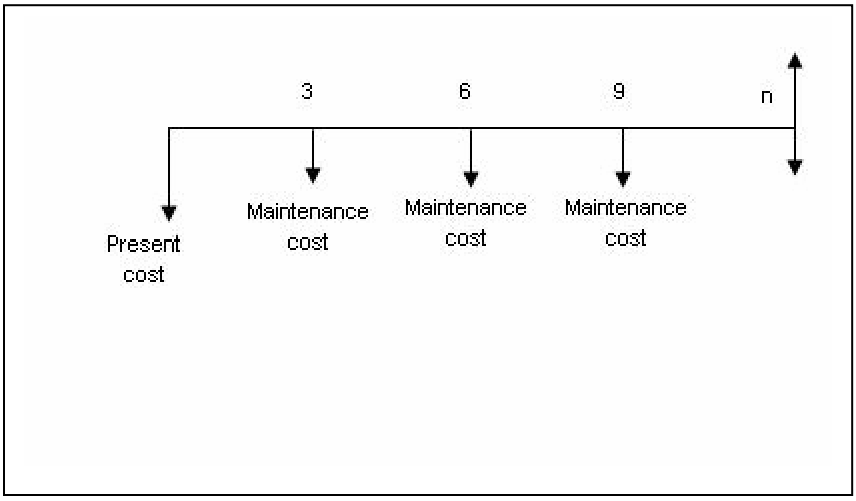


Figure5.4. Cash flow diagram

During investigating the probabilities when there is a choice in selecting different values for a variable, there must be a clear definition to the advantages and disadvantages of increasing or decreasing the value of that variable. According to the local condition, an optimum value should be selected. Generally, making a decision, through selecting one of available several alternatives, depends on the *experience* of the engineer. Here, the word experience is too flexible to be acceptable in engineering practice. Indirectly, this word indicates making a reasonable proper decision that is given with good intention according to the available circumstances of the project. Such decision is needed most when there is no clear data available for the design, or, when the problem has so many variables that can not be optimized mathematically.

In this case the optimization is performed mostly by the experience of the engineer or the designer. Sometimes it is difficult to create mathematical equations for future projects because it is not certain what will change in the future and how much of this change will affect the calculations. Such as, maintenance can not be written in mathematical form exactly despite the written ones can satisfy a little percentage of accuracy.

The maintenance cost equation of sewer line in this research is created by the help of the experiences of the engineers working in related water works establishments. Their experience in this field has helped the author to write such an equation. This equation may not be exactly true but it forms a cost value which has to be considered in the design. If it did not included in the project, this may mislead the designer during selecting the alternative combinations. The maintenance and future management operations should always be considered in infrastructure works because their costs are not cheap enough to despise.

In this research, the selection of the alternative solution is also on the interpretation of the designer or the user. The alternative combinations are presented in tabular form with their probable cost values also warning whether the budget is enough or not. The designer may select the cheaper one or may select the next expensive one due to his initiation on one of the variables. The cheaper alternative may have a low velocity than the expensive one so; he/she may prefer the expensive one due to his lived experience on velocity in the recent projects. In this case, the mathematically optimized results can just present the nominal approaches; the best selection is made through the combination of both nominal approaches and engineering experiences.

5.10.3. Comparison of SewDes 1.0 with other programs

SewDes 1.0 mainly uses the methods of The Bank of Provinces but the hydraulic calculations are done with equations rather than monograms. A software program could not find in the 8. District Directory of The Bank of Provinces located in Adana. In the recent years a student has submitted a simple program to the engineers of the establishment but generally the designs are prepared in the General Directory located in Ankara.

Mr. Çolpon Makeeva who has educated of master degree in Çukurova University has submitted a similar program written with FORTRAN but not presenting the alternative solutions with adequate combinations including cost estimations.

The comparison of available software programs having different restrictions may remove away the SewDes 1.0 from its objective where it enables for some flexibility regarding the availability of pipe diameters, and the choice between a deeper pipe with a smaller diameter or a shallower pipe with a larger diameter. Usages of the available software programs in the markets may constitute some drawbacks because the regulations are different in Turkey despite the hydraulic equations used in the design show a similarity with the others.

Pacheco and Gray have developed a program which subject to certain constraints will design a system that minimizes excavation and one which minimizes pipe size. The program does not ensure that the velocity at design flow is at least 0.6 m/s, but does offer a rapid method for preliminary design. The program listing could be modified to permit inclusion of partial flow calculations. SWMM[2] and others of the more sophisticated storm-water simulation models include provisions for modeling sanitary sewage flow, including its diurnal variations. Although not as economical as the model of Pacheco and Gray, SWMM[2] design option can be used to determine preliminary sizes for sanitary sewers by omitting the runoff and using sanitary sewage and infiltration. [5]

“FDOT Sewer Design” is also an alternative software program for design calculations where it can be free downloaded from http://cee.ucf.edu/software/. This program can also be used for combined systems due to its being computing the storm water discharge. There are also many such commercial programs in internet but approximately all of them are not free and adequate for Turkey standards.

In addition to the models discussed above, there are a large number of other systems with widely varying capabilities which are commercially available. Except for very small systems or for small additions or modifications to existing systems, it is unlikely to be economical to use manual calculation techniques in the design of sanitary sewers. [5]

SewDes 1.0 has a very easy usage. The population, water demand, sewer discharge are determined as first part and then the design calculations start. The program has the chance of forming a pipe market list for design. The alternatives of diameter and slope combinations are tabulated with their costs. The program ensures that the variables are used regarding the constraints. This first version (1.0) of the program works just for two manhole interval but capable of improvement for future versions in new researches. The properties and the usage manual are presented at the next section where all details are shown by print screens.

6. USER MANUAL OF SewDes 1.0

6.1. Introduction

This program is prepared for educational purpose where it is a master project in University of Gaziantep at Department of Civil Engineering. The program software is written by using *Visual Basic 6.0* version and compiled to *exe* file format in order to accommodate to common versions of MS.Windows 98, 2000, ME, XP software. Due to aim of the project, the program did not need necessity to use database. All variables and values are recorded to the temporary memory of computer.

6.2. Practice of Program with Visual Basic 6.0

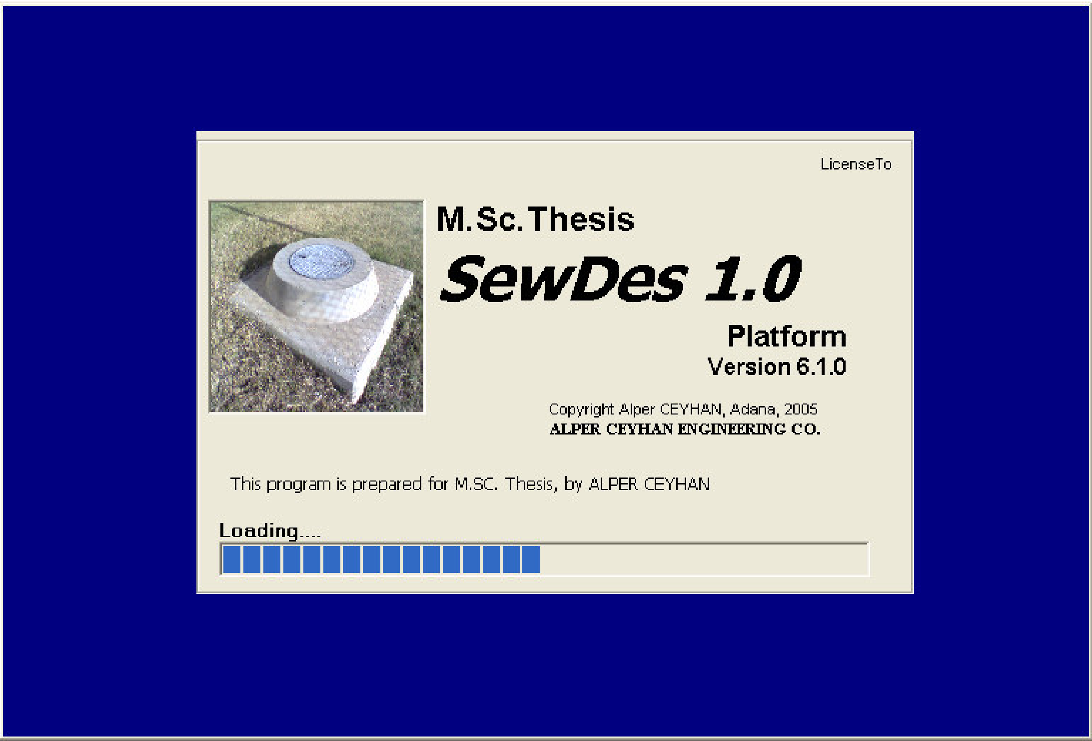


Figure 6.1 Loading process

The *SewDes 1.0* starts with the loading window which takes a few minutes. In this window: the name of the program, the version of the software and the name of the programmer are mentioned (Figure 6.1). This loading process starts by double clicking on "SewDes Visual Basic Program" named file where it is present in *SewDes 1.0* folder.

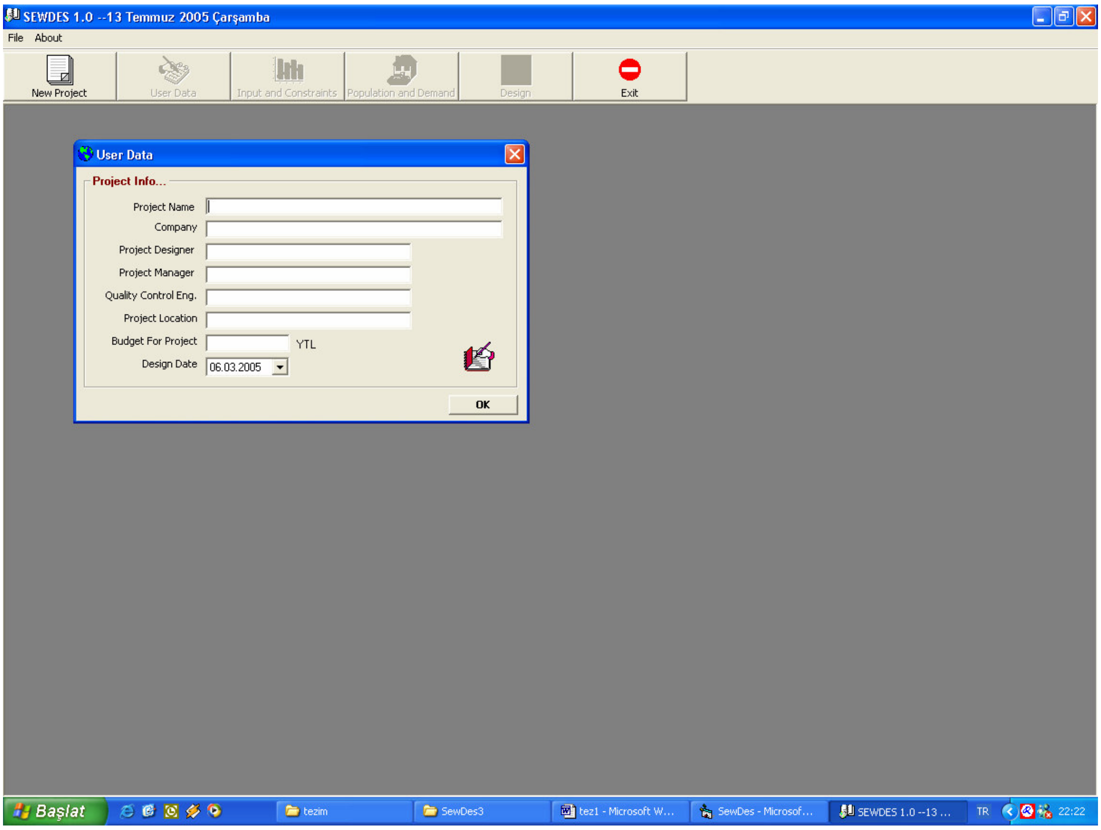


Figure 6.2 Main form of the Program

After the loading process has finished, the main form of the program appears as in the Figure 6.2. In this main form there exist six buttons located at the top of the frame where two of them are enabled and the others are disabled. This condition is provided in order to orient the user to correct usage. The program starts with a click on to the "New Project" button located at the upper left side. By the way a new project starts and the first user form appears as shown in Figure 6.3.

This "User Data" form has eight questions on it, all of which have to be filled at the beginning of the proposed design. As seen on Figure 6.3, the questions request the usual required information of a project. This type information generally exists on the reports and drawing sheets of projects. The design date can be selected easily where it refers to the starting date of design. In this form the most important input value is the budget amount, because the present cost estimation of the project illuminates whether the calculated outputs are adequate or not according to the money available. At the end of the design calculation, if the pipe cost and excavation cost totals exceed the budget input here then it is defined as "*Overload Budget*".

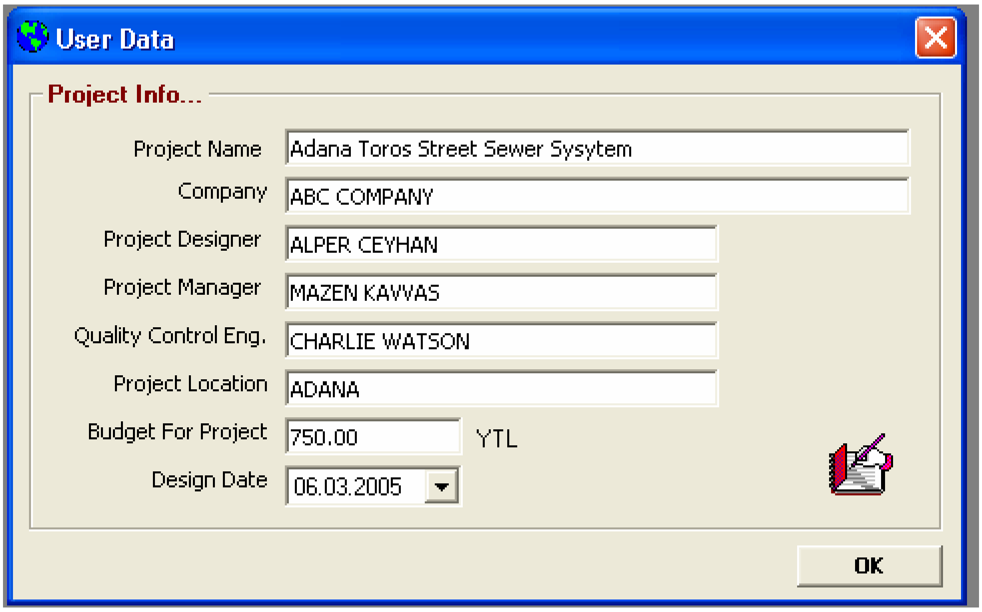


Figure 6.3 User Data form

If any of these questions are not answered then a message box appears to warn the user to fill the related statement. This warning method provides a correct usage without any missing. Fill all statements as in Figure 6.3 then press OK button.

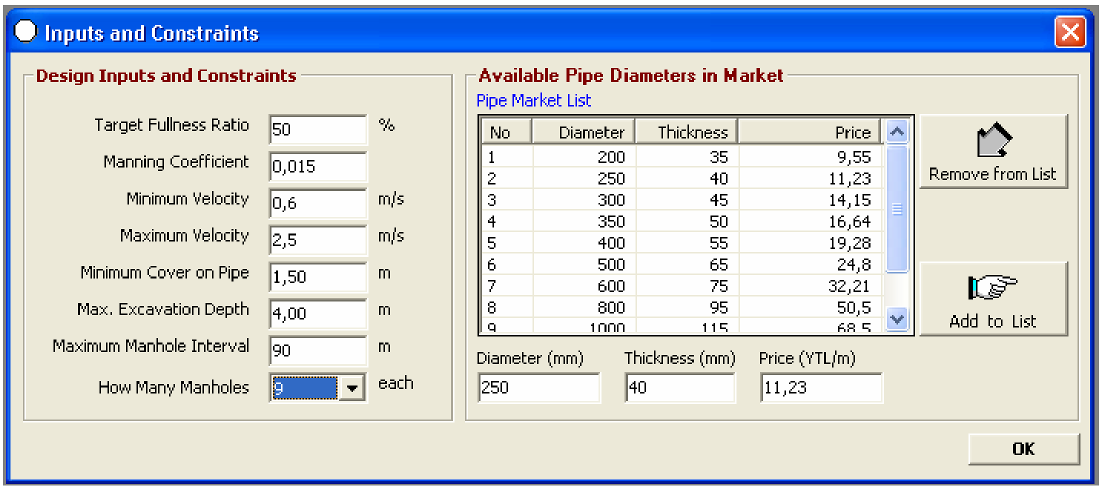


Figure 6.4 Input and constraints form

Prior to start to design calculations, the constraints are defined at this form and also a pipe market list is constituted (Figure 6.4). In this example, the values of pipe market list are taken from the 2005 dated Unit Price Book of the Ministry of Public Works of Turkey. The target fullness ratio is taken as %50 and the manning coefficient as 0.015. The warning method is also used in this form to prevent any data missing.

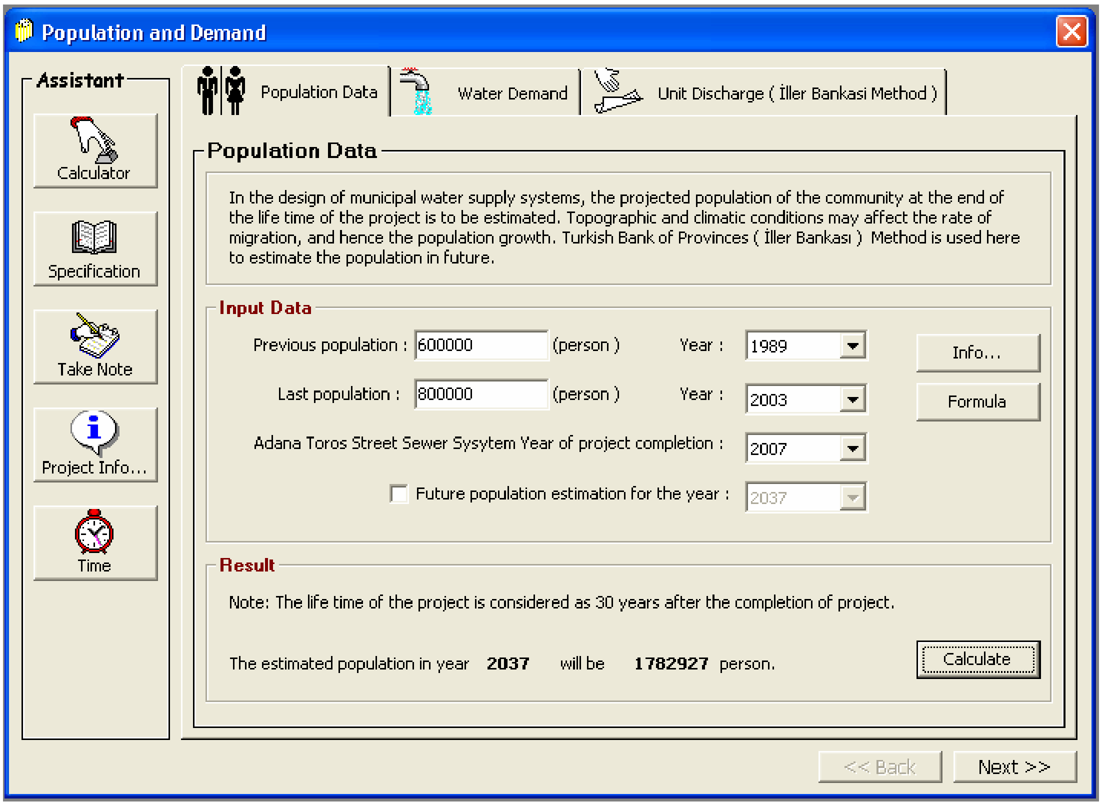


Figure 6.5 Population and demand form

After the constraints and market list has formed the "Population and Demand" form appears as in Figure 6.5 by clicking OK button. In this form the water demand and unit sewer discharge are calculated with respect to population. The formulas used and information may required during calculation so the program enables this help with related buttons. The future population is estimated with the formula of The Bank of Provinces and written in the result frame after clicking the "Calculate" button.

In this example:

The previous population = 600.000 person in year 1989

Last population = 800.000 person in year 2003

Year of project completion = 2007

The estimated population = 1.782.927 person in year 2037

Future population estimation year can be also changed by the user but in this example the acceptance is made according the specification and taken as 30 years.

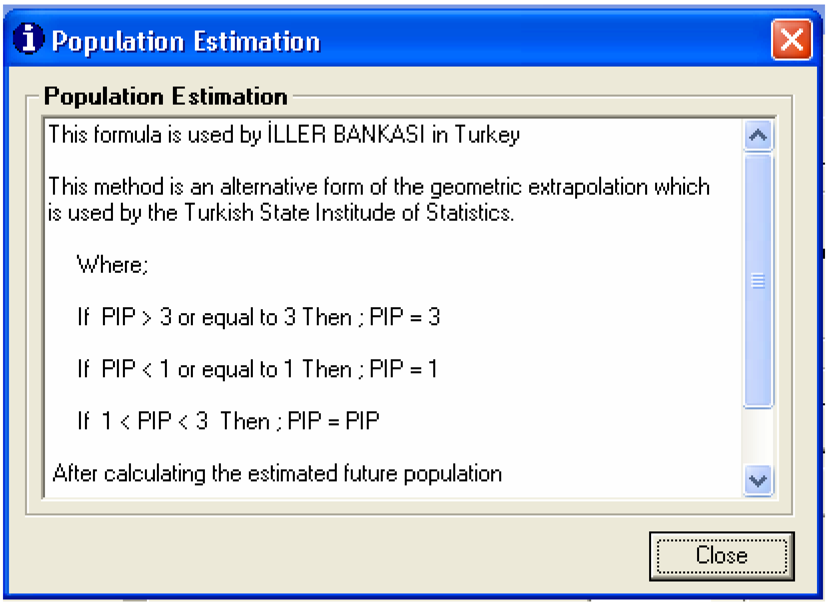


Figure 6.6 Information window

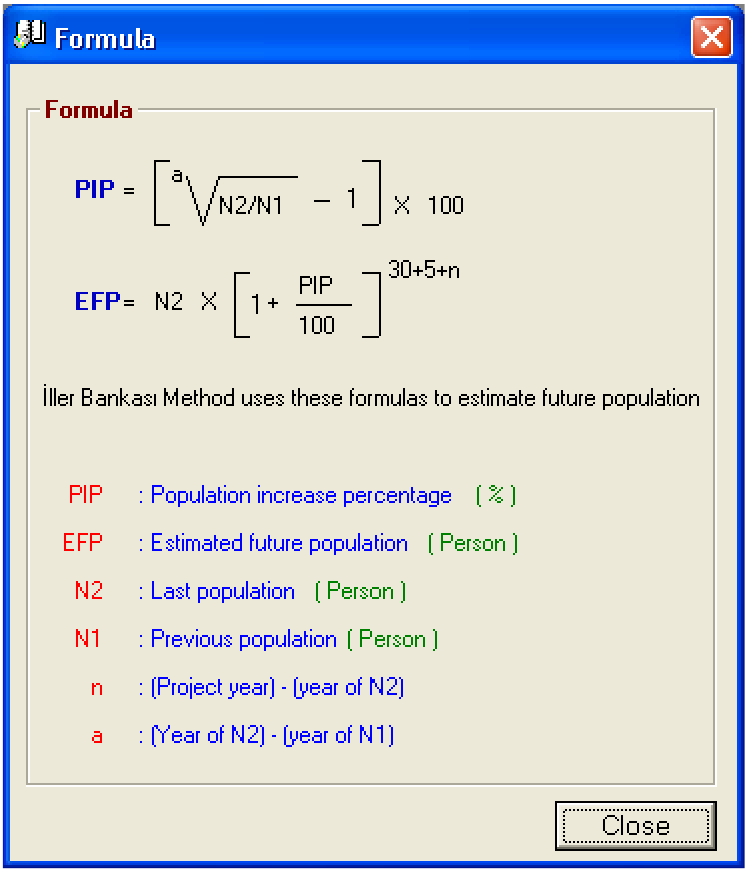


Figure 6.7 Formula window

The formula and information windows are as seen in Figure 6.6 and 6.7. It is easy to close these windows with the “Close” buttons located on them.

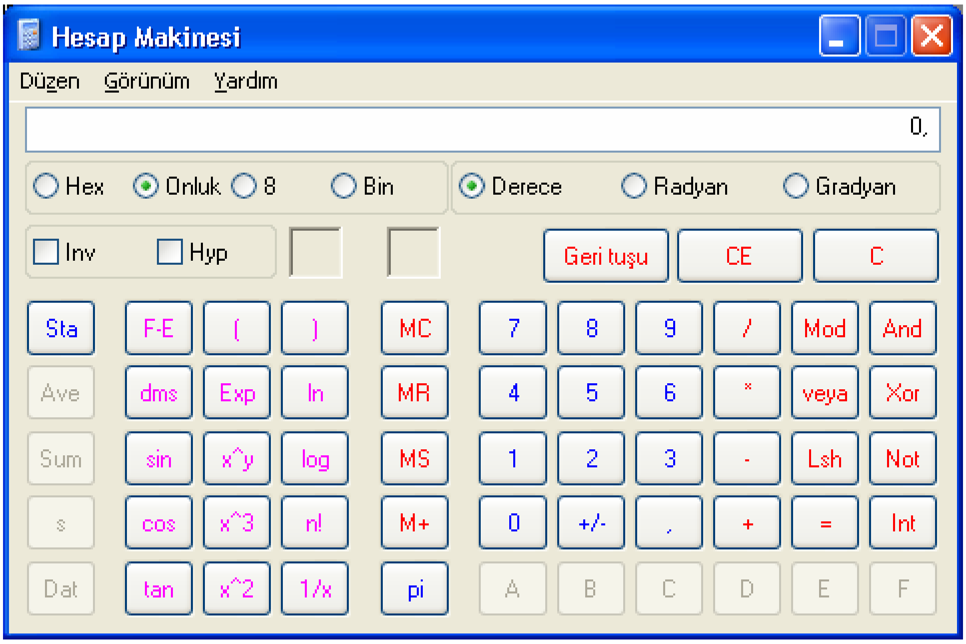


Figure 6.8 Calculators (Assistant)

Figure 6.9 Clocks (Assistant)

The "Assistant" frame located at the left side of the form is prepared in order to help to the user at least at calculating, time screening, specification reading, note taking and getting project information. The calculator and clock examples are shown in Figures 6.8 and 6.9 respectively. The specification button is not working at this moment because it has been not loaded in the program yet due to required official permissions from The Directorate of The Bank of Provinces and also the specification content varies according to the establishment selected so it is better to determine it by the designer or the user. This situation does not affect the aim of the thesis and the program so the duty of these buttons in the assistant frame can be improved for another version of this program in the future by having a different project aim.

Hopefully, this "Assistant" frame is created to give a nice point of view to *SewDes 1.0* regarding its being a *visual* program as its software language.

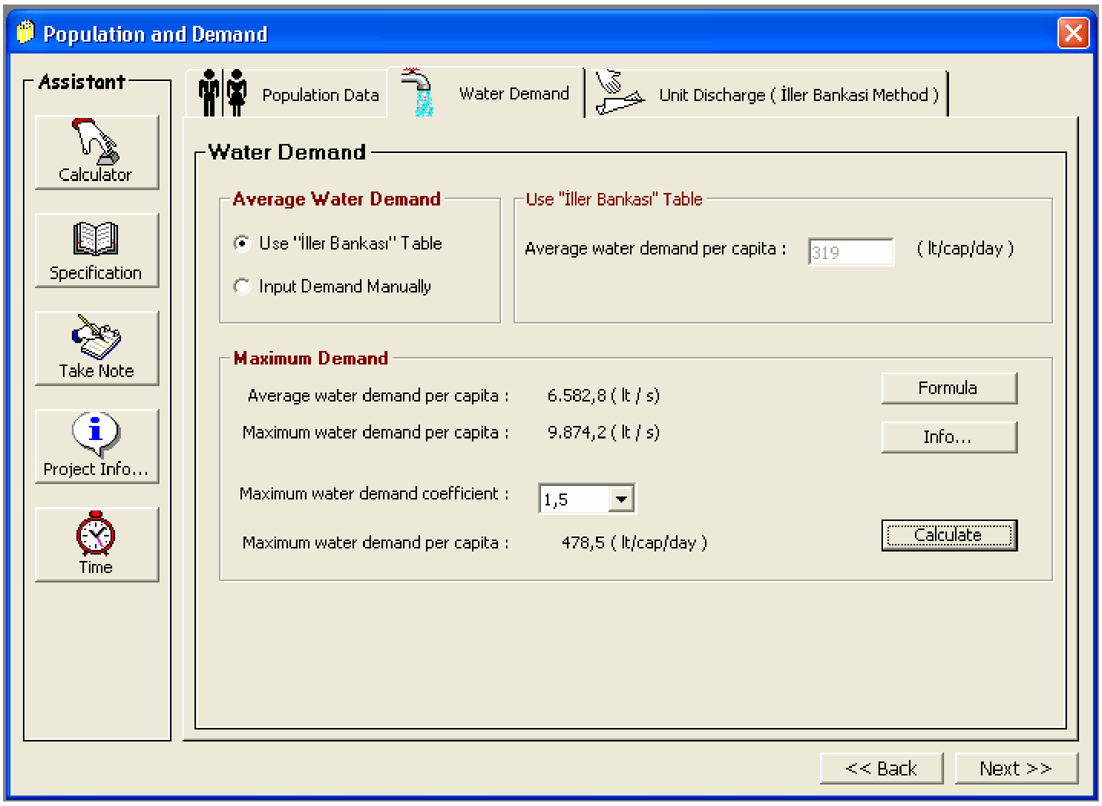


Figure 6.10 Water demand calculation

When the future population has estimated, the water demand value is calculated in this form which can be seen also by clicking the “Next” button at the bottom or the “Water Demand” button at the top of the form. The “Formula” and “Information” buttons also exist here to provide the probable necessary view.

The “İller Bankası” (The Bank of Provinces) alternative is selected for this example thus the average water demand per capita value with respect to population is calculated automatically by the program using its related tables where is calculated as 319 Lt/cap/day for 1.782.927 person.

The maximum water demand coefficient is selected as 1, 5. The calculated result is written at the bottom of the frame as 478, 5 Lt/cap/day after clicking the “Calculate” button. The (Lt/s) water demand values are also available in this form but due to computations being performed by the unit of Lt/cap/day, it is recommended to use 478, 5 Lt/cap/day value for hand made calculations.

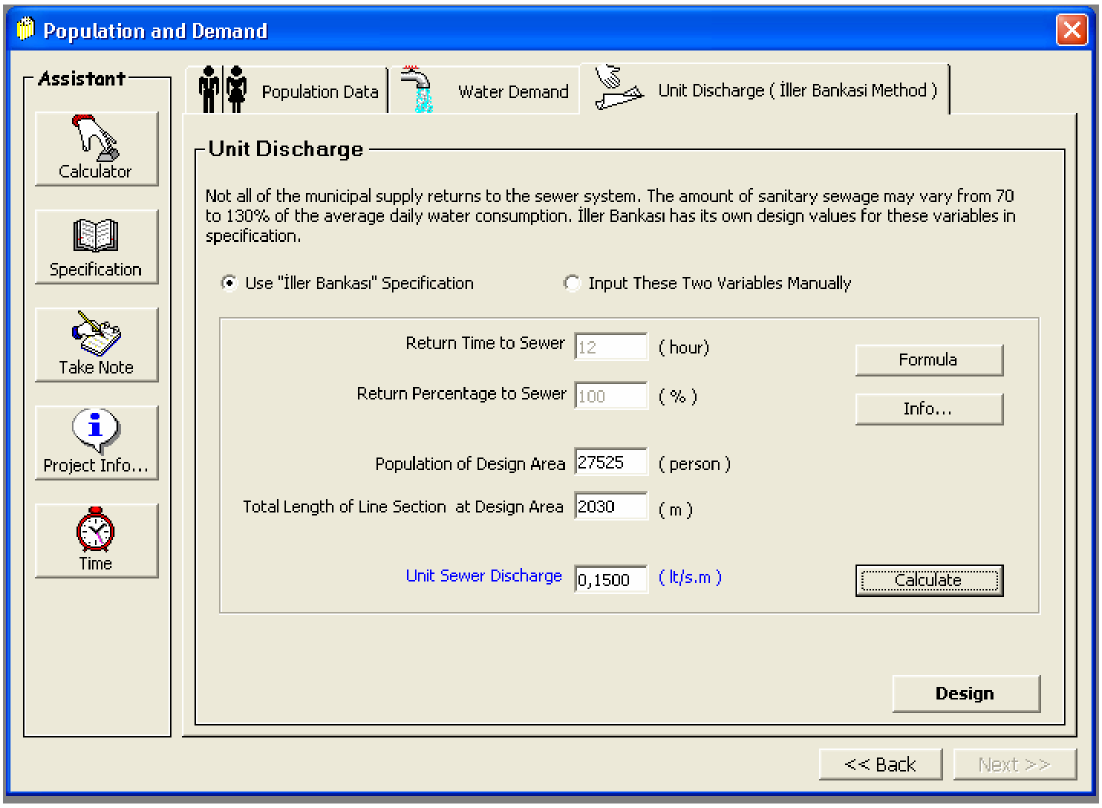


Figure 6.11 Unit sewer discharge

The return time and return percentage is selected as 12 and 100 respectively. These values are the acceptances of The Bank of Provinces so when it is selected as in figure 6.11 these variables are designated automatically and can not be change by the user. If different values for these variables are required then the “Input These Two Variables Manually” button should be selected.

Equation (4.8) is used to calculate the unit sewer discharge. The variables are:

Return time to sewer = 12 hour

Return percentage to sewer = % 100

Population of design area = 27.525 person

Total length of section = 2030 m.

Thus;

The calculated unit sewer discharge is 0, 15 lt/s.m

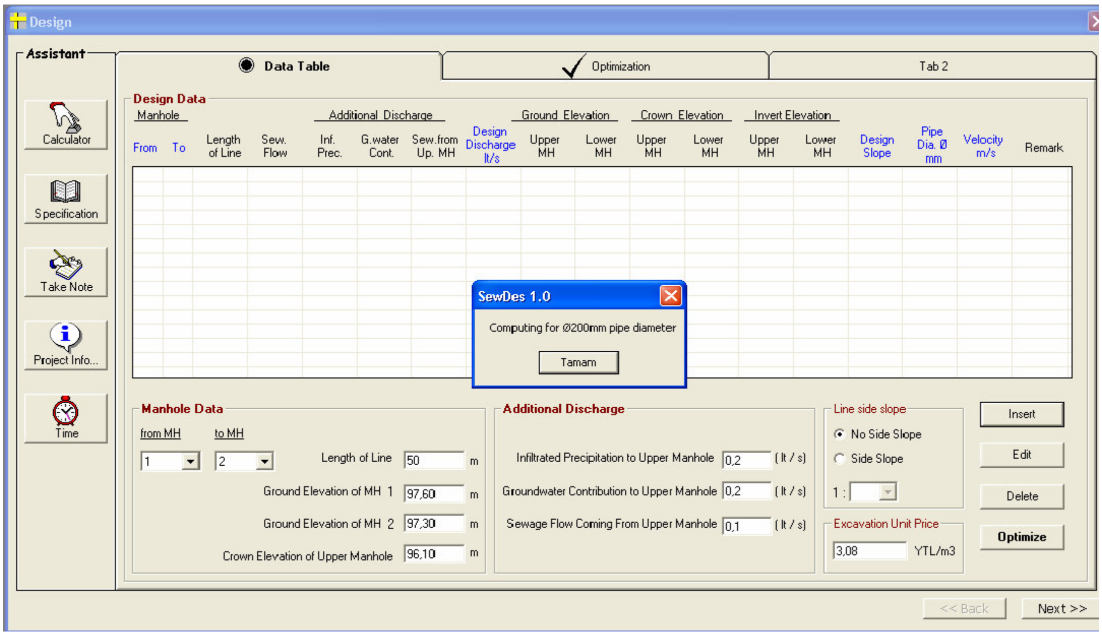


Figure 6.12 Hydraulic design tables

The manhole numbers with ground elevations are written in this form. Also the length of line between these manholes and the crown elevation of upper manhole is written as design data in this form.

The additional discharges as infiltrated precipitation to upper manhole, groundwater contribution to upper manhole and the sewage flow coming from upper manhole are input as 0,2 lt/s, 0,2 lt/s, 0,1 lt/s respectively.

Side slope is not used for this example; it is accepted as the trench is excavated in rectangular shape due to soil being hard and stable. Anyway, if the trench requires a side slope it can be selected with the side slope button having various alternatives.

The excavation unit price is accepted as 3.08 YTL/m^3 where it taken from the 2005 press Unit Price Book of The Ministry of Public Works Establishment. The unit price of excavation changes according to type of soil and depth of excavation so the unit price input area is presented to the user or the designer.

Thereafter inputting the variables the "Insert" button is clicked and the hydraulic computation starts. The program computes a pipe diameter according to the line slope which is determined according to the conditions mentioned in section 5.6.

The first adequate pipe diameter available in the market list is taken in consideration and computations are performed regarding the minimum velocity. The selected pipe diameter and calculated flow velocity are written on a message box as seen in Figure 6.12.

The program started to compute for Ø200 mm diameter. The velocity due to this diameter and selected slope (which is 0.0060 for this example) is computed as 0.562 m/s where it is less than the minimum velocity value. The constraint for the minimum velocity was 0.6 m/s, in this case the program will go to pipe market list and select the next larger pipe diameter to re-compute. This loop will continue until the pipes finish at the market list.

The program tried the other next pipe in the market list and found Ø250 mm pipe diameter. The new velocity computed, becomes 0.652 m/s with this Ø200 mm diameter. This situation is written in message box firstly and then tabulates in the design table as seen in Figure 6.13 by the approval of the user.

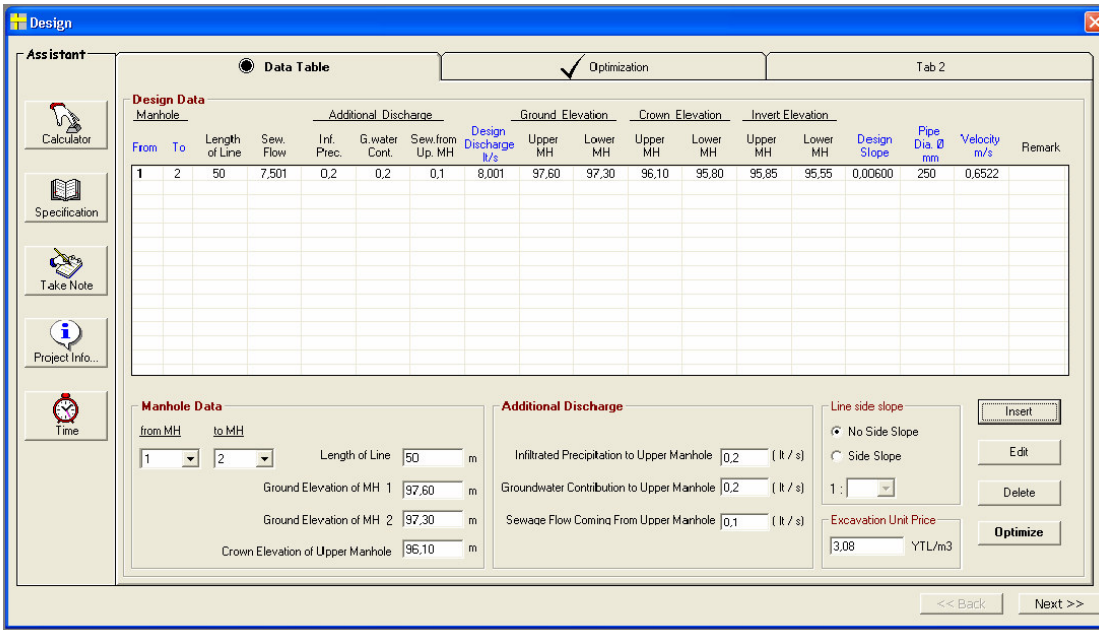
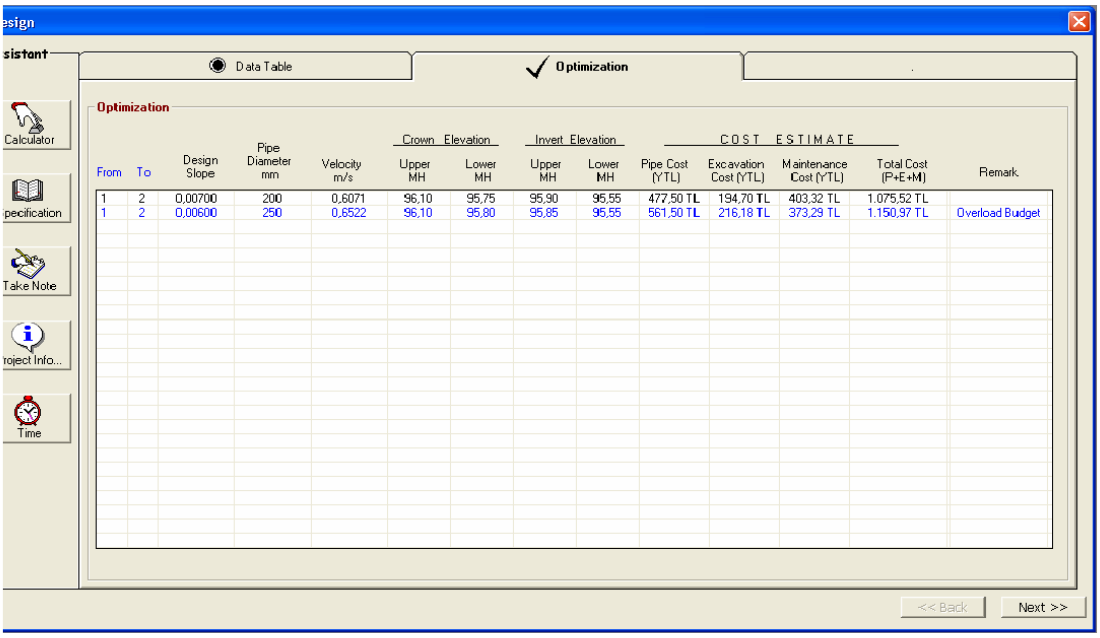


Figure 6.13 Tabulated forms of variables and outcomes

As seen in the figure above, the input and outcome values are written in the design table providing the minimum velocity requirements. Here, the first diameter Ø200 mm was not adequate enough to satisfy the velocity condition with the accepted line slope. Financially, considering the pipe prices it is more expensive to use Ø250 mm pipe than using Ø200 mm pipe. Remembering the aim for optimization, the cost of results has to be minimized. In this case an optimization is required to determine the minimum priced result among the available alternatives. *SewDes 1.0* performs optimization by presenting the alternative solutions which are formed by the various adequate combinations of line slope and pipe diameter regarding their affects on cost estimations.



From	To	Design Slope	Pipe Diameter mm	Velocity m/s	Crown Elevation Upper MH	Crown Elevation Lower MH	Invert Elevation Upper MH	Invert Elevation Lower MH	COST ESTIMATE Pipe Cost (YTL)	COST ESTIMATE Excavation Cost (YTL)	COST ESTIMATE Maintenance Cost (YTL)	COST ESTIMATE Total Cost (P+E+M)	Remark
1	2	0,00700	200	0,6071	96,10	95,75	95,90	95,55	477,50 TL	194,70 TL	403,32 TL	1.075,52 TL	
1	2	0,00600	250	0,6522	96,10	95,80	95,85	95,55	561,50 TL	216,18 TL	373,29 TL	1.150,97 TL	Overload Budget

Figure 6.14 Optimization results

When the "Optimization" button is pressed, the table in Figure 6.14 is displayed. Here, the possible combinations having different pipe diameter and line slope are tabulated with their present installation cost and future maintenance cost estimations. The present cost estimation value is also compared with the budget available and the conclusion is written as "Overload Budget".

Now it is up to designer to decide which alternative will be selected among the existing combinations. Although it is being common to choose the cheaper one, sometimes the engineering initiation causes a change in the conditions with respect to experience. Designer may prefer Ø250mm pipe diameter with 0.006 slope value despite its higher present cost. However, the judgment for selection may not only be in accordance with the present installation cost but also may be related with the expected probable future maintenance cost. Furthermore the decision may be given according to the calculated value of design velocity where the designer may chose the one alternative which has higher velocity and higher cost. The judgment in *SewDes 1.0* does not have any experience contribution, the optimized solutions regarding the estimated costs are just presented and it is aimed to use the cheapest one.

In this example the present installation cost and total cost of Ø200 mm pipe is cheaper than the Ø250 pipe, despite the maintenance cost of Ø200 mm pipe is higher than Ø250 pipe. Regarding the budget; the Ø200 mm pipe with %0.7 slope is preferred by *SewDes 1.0.*

MANHOLE		HYDRAULIC OUTPUTS			ELEVATIONS				COST ESTIMATION				
From	To	Design Slope	Pipe Diameter (mm)	Design Velocity (m/s)	Upper MH Crown Elv.	Lower MH Crown Elv.	Upper MH Invert Elv.	Lower MH invert Elv.	Pipe Cost (YTL)	Exca. Cost (YTL)	Maint. Cost (YTL)	Total Cost (YTL)	Remark
1	2	0,007	200	0,6071	96,10	95,75	95,90	95,55	477,50	194,70	403,32	1.075,52	
1	2	0,006	250	0,6522	96,10	95,80	95,85	95,55	561,50	216,18	373,29	1.150,97	Overload Budget

Figure 6.15 Tabular forms of optimization results

Table in Figure 6.15 illustrates the variables and outcomes in a tabular form, where it is easily seen that the present installation cost (Pipe Cost + Excavation Cost) of Ø250 mm pipe (Blue line) overloaded the budget.

Due to computation started from Ø250 mm pipe (Initial pipe diameter), the other possible combinations are performed with the pipes which are smaller than the initial one and available in the pipe market list. If the initial pipe diameter were Ø400 mm then the combinations should be for performed with the diameters Ø350, Ø300, Ø250 and Ø200 mm.

As a recommendation, all outputs of any computer program should be checked by an experienced engineer prior to use the results on a real system construction.

6.3. Practice with Manual Calculating

Here, the same example computed above will be solved without computer program. All input values will be taken as same and a hand-calculator will be used for calculating operations.

Table 6.1 Input and constraints

Target Fullness Ratio	=	50	%
Manning Coefficient	=	0,015	
Minimum Velocity	=	0,6	m/s
Maximum Velocity	=	2,5	m/s
Minimum Cover over Pipe	=	1,5	m
Maximum Excavation Depth	=	4	m
Maximum Manhole Interval	=	90	m
Number of Manholes	=	9	each

Table 6.2 Pipe market list

Pipe Diamater (mm)	Pipe Thickness (mm)	Unit Price (YTL/m)
200	35	9,55
250	40	11,23
300	45	14,15
350	50	16,64
400	55	19,28
500	65	24,8
600	75	32,21
800	95	50,5
1000	115	68,5
1200	120	85,4

The previous population = 600.000 person in year 1989

Last population = 800.000 person in year 2003

Year of project completion = 2007

a = 2003-1989

a = 14 year

Using the equation (4.1) population increase percentage is calculated as:

$$P = \left(\sqrt[14]{\frac{800000}{600000}} - 1 \right) \times 100$$

P = 2.0761298 here $1 < P < 3$ then P=2.0761298

Using the equation (4.2) the future population is calculated as:

$$N_g = 800000 \times \left(1 + \frac{2.0761298}{100} \right)^{30+5+4}$$

N_g = 1.782.927 person (Estimated future population)

Using the table (4.3) and by interpolation average water demand is calculated as:

1.000.000 person 280 lt/cap/day

1.782.927 person *X* lt/cap/day

2.000.000 person 330 lt/cap/day

X = 319, 15 lt/cap/day

Maximizing coefficient (c) = 1, 5

Maximum water Demand = 319, 15 lt/cap/day x 1, 5

Maximum water Demand = 478, 7 lt/cap/day

The given information:

Population in design area = 27525 person

Total length of line at design area = 2030 m

Return time to sewer = 12 hour

Return percentage to sewer = %100

Using the equation (4.8) the unit sewer discharge is calculated as:

$$Q = \frac{27.525 \times 478.7 \times 1}{12 \times 2030 \times 3600}$$

$$Q = 0,\ 150 \text{ lt/s.m}$$

Using the manhole interval length in table (6.1) and equation (4.9) the design discharge is calculated as:

The given information:

Infiltrated precipitation to upper manhole = 0, 2 lt/s

Groundwater contribution to upper manhole = 0, 2 lt/s

Sewage flow coming from upper manhole = 0, 2 lt/s

$$Q_{dsgn} = (0{,}15 \times 50) + (0{,}2 + 0{,}2 + 0{,}1)$$

$$Q_{dsgn} = 8 \text{ lt/s}$$

$$Q_{dsgn} = q$$

Using the Fullness Ratio in Table 6.1 and putting it into Figure5.2,

$$\frac{q}{Q_{full}} = 0{,}41 \quad \text{And} \quad \frac{v}{V_{full}} = 0{,}80$$

$$Q = \frac{8}{0{,}41} = 19{,}512 \text{ lt/s}$$

$$Q = 0{,}019 \text{ m}^3\text{/s}$$

Using the equation (5.1a) the diameter of pipe is calculated as:

$$D = 2 \times \left(\sqrt{\frac{0{,}019}{0{,}6 \times \pi}} \right)$$

$$D = 0{,}200 \text{ m}$$

Using the equation (5.6b) the minimum and maximum slope are calculated as:

$$S_{min} = \left[\frac{0{,}6 \times 0{,}015}{\left(\frac{0{,}100}{2}\right)^{2/3}} \right]^2$$

$S_{min} = 0{,}00440$

$$S_{min} = \left[\frac{2{,}5 \times 0{,}015}{\left(\frac{0{,}100}{2}\right)^{2/3}} \right]^2$$

$S_{max} = 0{,}07634$

Using the equation (5.5) the ground slope is calculated as:

$$S_{grnd} = \frac{97{,}60 - 97{,}30}{50}$$

$S_{grnd} = 0{,}00600$

a) $S_{min} < S_{grnd} < S_{max}$

b) Cover depth = 97,60 - 96, 10
Cover depth = 1,50 m

Due to condition of (a) and (b) the ground slope is selected as line slope (0, 00600)

Putting equation (5.6) into equation (5.1a) the initial diameter is calculated as:

$$D = 2 \times \left[\frac{0{,}019 \times 0{,}015}{\pi \times \sqrt{0{,}00600} \times (0{,}5)^{2/3}} \right]^{3/8}$$

D = 0, 1892 m then choose Ø200 mm pipe from pipe market list

Note: The diameter is also found as 175 mm by using the Manning Monogram.

Using the equation (5.6) the full flow velocity is calculated as:

$$V_{full} = \frac{1}{0,015} \times \left(\frac{0,100}{2}\right)^{2/3} \times (0,00600)^{1/2}$$

$$V_{full} = 0,700 \text{ m/s}$$

$$\frac{v}{V_{full}} = 0,80 \qquad \text{Was obtained from Figure 5.2}$$

Design velocity is calculated as:

$$v = 0,70 \times 0,80$$

$$v = 0,56 \text{ m/s}$$

The calculated design velocity is less than minimum velocity so the next larger pipe available in the market list will be tried in design to satisfy the minimum velocity requirement. The next larger pipe in the market is Ø250 mm thus it will be tried in place of initial diameter.

$$V_{full} = \frac{1}{0,015} \times \left(\frac{0,125}{2}\right)^{2/3} \times (0,00600)^{1/2}$$

$$V_{full} = 0,813 \text{ m/s}$$

New design velocity is calculated as:

$$v = 0,813 \times 0,80$$

$$v = 0,6506 \text{ m/s}$$

The calculated new design velocity is bigger than minimum velocity so it is acceptable to use. The installation elevations and cost can be calculated using this accepted pipe diameter.

Using the equations (5.21, 5.22, 5.23) the elevations are calculated as:

$$E_{lc} = 96,10 - (50 \times 0,00600)$$

$$E_{lc} = 95,80 \text{ m}$$

$$E_{ui} = 96,10 - 0,250 = 95,85 \text{ m}$$

E_{li} = 95,80 – 0,250 = 95,55 m

Excavation amount is calculated by using the average value of cover depths in trench section area and multiplying it by trench width as:

Ø250 mm pipe thickness = 40 mm
Sand bedding under pipe = 0, 133 m (using equation 5.25)

Excavation depth of upper side:

$$H = 97{,}60 - 95{,}85 + 0{,}04 + 0{,}1330$$

$$H_u = 1{,}923 \text{ m}$$

Excavation depth of lower side:

$$H = 97{,}30 - 95{,}55 + 0{,}04 + 0{,}1330$$

$$H_l = 1{,}923 \text{ m}$$

Then the average excavation height of trench section is 1, 92 m

The bottom width of the trench is calculated by the equation (5.24)

$$B = 0{,}25 + 0{,}04 + 0{,}04 + 0{,}20 + 0{,}20$$

$$B = 0{,}73 \text{ m}$$

The excavation amount is:

$$E = 1{,}923 \times 0{,}73 \times 50$$

$$E = 70{,}190 \text{ m}^3$$

Cost estimations are calculated by using the 2005 year version Government Unit Prices. Excavation unit price is taken as 3.08 YTL, and unit price of Ø250 pipe is 11.23 YTL (Table 6.2).

Pipe Cost = 50 x 11.23 = 561.50 YTL
Excavation Cost = 70,190 x 3.08 = 216.18 YTL
Total Installation Cost = 777.68 YTL (Overload Budget, Budget = 750 YTL)

Maintenance cost estimation is performed assuming the 3 m length of pipe for repairing and using the repetition year N as 4 years for Ø250 mm pipe.

$$\text{Maintenance Cost} = 3 \times \left(\frac{777.68}{50}\right) \times \left(\frac{30}{4}\right)$$

Here, the (30 / 4) value will be taken as 8, because it is programmed to round this division to upper.

Maintenance Cost =373.29 YTL

<u>Optimization</u>

Optimization is made here by using the smaller pipe (Ø200mm) with steeper slope. Steeper slope is obtained by trial method, the first obtained line slope is increase by adding 0.0005 and then it continued until getting an adequate velocity. The results are then tabulated in a table. The optimization is made with respect to pipes available in market list so if the diameter of pipes in the market changes, the alternative combinations change naturally.

Due to finding of *SewDes 1.0* the initial diameter as Ø250 mm then the optimization will be tried for Ø200 mm diameter.

$$V_{full} = \frac{1}{0,015} \times \left(\frac{0,100}{2}\right)^{2/3} \times (0,00650)^{1/2}$$

$V_{full} = 0,729$ m/s

v = 0,583 m/s Where v < 0, 6; so increase slope again

$$V_{full} = \frac{1}{0,015} \times \left(\frac{0,100}{2}\right)^{2/3} \times (0,0070)^{1/2}$$

$V_{full} = 0,757$ m/s

v = 0,605 m/s Where v >0, 6; so adequate velocity

The alternative combination with optimization is found as Ø200 mm diameter pipe with a slope of %0, 7.

Using the equations (5.21, 5.22, 5.23) the optimized elevations are calculated as:

$$E_{lc} = 96,10 - (50 \times 0,00700)$$

$$E_{lc} = 95,75 \text{ m}$$

$$E_{ui} = 96,10 - 0,200 = 95,90 \text{ m}$$

$$E_{li} = 95,75 - 0,200 = 95,55 \text{ m}$$

Excavation amount is calculated in the same manner

Ø200 mm pipe thickness = 35 mm

Sand bedding under pipe = 0, 127 m (using equation 5.25)

Excavation depth of upper side:

$$H = 97,60 - 95,90 + 0,035 + 0,127$$

$$H_u = 1,862 \text{ m}$$

Excavation depth of lower side:

$$H = 97,30 - 95,55 + 0,035 + 0,127$$

$$H_l = 1,912 \text{ m}$$

Then the average excavation height of trench section is 1, 887 m

The bottom width of the trench is calculated by the equation (5.24)

$$B = 0,20 + 0,035 + 0,035 + 0,20 + 0,20$$

$$B = 0,67 \text{ m}$$

The excavation amount is:

$$E = 1,887 \times 0,67 \times 50$$

$$E = 63,21 \text{ m}^3$$

Cost estimations are calculated by using the 2005 year version Government Unit Prices. Excavation unit price is taken as 3.08 YTL, and unit price of Ø200 pipe is 9.55 YTL (Table 6.2).

Pipe Cost = 50 x 9.55 = 477.50 YTL

Excavation Cost = 63.210 x 3.08 = 194.69 YTL

Total Installation Cost = 672.19 YTL (Less than Budget, Budget = 750 YTL)

Maintenance cost estimation is performed assuming the 3 m length of pipe for repairing and using the repetition year N as 3 years for Ø200 mm pipe.

$$\text{Maintenance Cost} = 3 \times \left(\frac{672.19}{50}\right) \times \left(\frac{30}{3}\right)$$

$$\text{Maintenance Cost} = 403.31 \text{ YTL}$$

The results of manual calculation may display a little difference in values due to number of digits taken in consideration where the computer uses all digits after point in computing on the other hand manual calculation does not.

Table 6.3 Results in tabular form

MANHOLE		HYDRAULIC OUTPUTS			ELEVATIONS				COST ESTIMATION				
From	To	Design Slope	Pipe Diameter (mm)	Design Velocity (m/s)	Upper MH Crown Elv.	Lower MH Crown Elv.	Upper MH Invert Elv.	Lower MH invert Elv.	Pipe Cost (YTL)	Exca. Cost (YTL)	Maint. Cost (YTL)	Total Cost (YTL)	Remark
1	2	0,007	200	0,6071	96,10	95,75	95,90	95,55	477,50	194,69	403,31	1.075,50	
1	2	0,006	250	0,6522	96,10	95,80	95,85	95,55	561,50	216,18	373,29	1.150,97	Overload Budget

As it is seen from the Table (6.3) the Ø200 mm pipe combination which means the smaller diameter pipe with steeper slope costs cheaper than the combination of larger diameter pipe with shallower slope alternative.

7. CONCLUSIONS

The conveyance of sewage which is provided by sewer system includes not only physical structures required but also includes a design study where this study enables the system either more productive or unproductive. The sensitiveness of the design study for sewer systems affects both the life continuity and the total cost of the projects to be prepared.

Sewer systems are designed to meet the projected future demand with technical and economical feasibility. Since the future demand is depending upon the future population; a well estimated population value should be considered in design studies. The accuracy and the reliability of the other design variables also constitute an important role in the achievement of technical and economical aspects.

Conventional methods used for the design and analysis are based on iterative trial error procedures which are commonly performed by manual calculations. In order to minimize the computational efforts, beside the computer aided programs, engineering judgment and experience are required. Therefore, the designer must be well conversant with the characteristics of the system. In this case optimization is the best method to obtain adequate outcomes in a shorter time. Optimization eliminates the trial-error process and yields a unique solution regarding the constraints.

The sewer system designs due to having so many variables require a careful decision during the selection of input and output values. Different combinations of various results may provide more adequate solutions regarding technical and economical aspects. The different combinations of pipe diameter and line slope are discussed in this study and optimized results having cost estimates are presented to the designer. A Visual Basic Program that enables for some flexibility regarding the availability of pipe diameters, and the choice between a deeper pipe with a smaller diameter or a shallower pipe with a larger diameter is compared by using the existing pipe diameters that are determined by the user.

The program achieves a sewer system design starting from population estimation and calculation of sewer discharge where it presents a visual display of results with changing variables. The program named *SewDes 1.0* also reduces the time consumption formed by the manual hydraulic calculations. Due to its having the aim of presenting the alternative solutions with respect to pipe diameter and slope, *SewDes 1.0* has been programmed not to be used as a commercial program like the others in the markets where data base recording is not used in this program and the software can not be compared with the commercial ones.

As a result, *SewDes 1.0* presents a sewer design which investigates the probabilities when there is a choice in selecting different values for a variable in sewer system regarding the constraints and presents the most adequate alternative having the minimum cost.

REFERENCES

1- Departments of the U.S Army, and the Air Force. (1985). Sanitary *and Industrial Wastewater Collection - Gravity Sewers and Appurtenances.* Air Force AFM 88-11, Volume 1

2- Yanmaz, A. M., (1997). *Applied Water Resources Engineering*. Metu Press

3- Samsunlu, A., (1997). *Su Getirme ve Kanalizasyon Yapılarının Projelendirilmesi*. (7th Ed.) Sam-Çevre Teknolojileri Merkezi Yayınları

4- Linsley, R.K., Franzini, J. B., Freyberg, D. L., and Tchobanoglous, G., 1992. *Water Resources Engineering,* Singapore: Mc Graw Hill.

5- Terence J. McGhee. (1991). *Water Supply and Sewerage*. (6th Ed.) Singapore: Mc Graw Hill

6- Kavvas, M., (2002). An Appraisal of Sewer System Problems in Developing Countries. *Elsevier,* 16, 1366-7017

7- Makeeva, Ç., (1998). *Bilgisayar Yardımı ile Kanalizasyon Şebekelerinin Hidrolik Tasarımı*. Çukurova Üniversitesi Fen Bilimleri Enstitüsü İnş. Müh. ABD

8- STBP, 1991. *Specifications for Sewer System Projects,* Ankara (in Turkish): Turkish Bank of Provinces

9- Ceyhan, A., (2000). *Water Distribution System in Gaziantep*. University of Gaziantep Faculty of Engineering Department of Civil Engineering

10- NCPI (27june 2005) -, http:/www.ncpi.org *Engineering Manual, Chapter 2*

11- Ling, D. J., et al. (1989). *Social costs of sewerage rehabilitation –where can no-dig techniques help? Tunneling and Underground Space Technology*, 4(4), 495-501

12- Chow, V.T., 1959. “Open Channel Hydraulics”, New York: McGraw Hill

13- Jean Nougaro : *Theoretical and experimental study of the propagation of translation waves in open channels* No.284, 1953

14- J.Nougaro : *Graphical method for the computation of the propagation of translatory waves in open channels*, Proceedings of the 6th General Meeting, International Association for Hydraulic Research, The Hague 1955, vol. 4, pp. D5-1 to D5-15,1955

15- Okka, O., (2000). *Mühendislik Ekonomisi*. (3rd Ed.) Ankara: Nobel Yayın