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ALTERNATIVE PATH ON SEWERAGE SYSTEM; CONDOMINIAL METHOD AND ITS APPLICATION

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To My Family

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Foremost and More over I want to pay special regards to my family who are enduring these all expenses and supporting me in all events. I am nothing without their prayers. They also encouraged me in crises. I shall never forget their sacrifices for my education so that I can enjoy my successful life as they are expecting, I will never forget my father, my mother, my brother and my sisters. They may get peaceful life in Heaven.

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I hope this thesis proves worthy of your trust and hope that it can be useful for other student to have information they need from it.

ABSTRACT

Simplified sewerage is an off-site sanitation technology that removes all wastewater from the household environment. Conceptually it is the same as conventional sewerage, but with conscious efforts made to eliminate unnecessarily conservative design features and to match design standards to the local situation.

Most of the new technologies are developed and designed to deal with health and environmental concerns. Especially in third world countries, the unplanned urban, development result in divesting wastewater collecting and treating problems.

In this study, the fundamental of theory of simplified sewer design is discussed by the help of readily available computer program called; "Pc-based simplified sewer design" is used to analyze the effect of the sewer diameters and gradients for different cases. The program aims to speed up the design calculation, provide different design configurations at a short time interval, and helps those people whose purpose is training or learning of simplified sewer design.

This thesis provides a detailed study of the Simplifying sewerage system.

LIST OF SYMBOLS

q	Daily peak flow rate (wastewater flow), l/s
K_1	Peak factor
K_2	Return factor
P	population served by length of sewer under consideration
w	Average water consumption, L/ capita / day
a	Area of flow, m^2
p	Wetted perimeter, m
r	Hydraulic radius, m
b	Breadth of flow, m
θ	Angle of flow, expressed in radians
d	Depth of flow, m
D	Sewer diameter, m
T	Temperature,
I	Dimensionless sewer gradient, m/m
v	Velocity of flow, m/s
n	Dimensionless Ganguillet-Kutter roughness coefficient
τ	Tractive tension (shear stress), N/m^2
W	Weight and the volume of the sewer, N
L	Length of sewer, m
ρ	Density of wastewater, kg/m^3
g	Gravitational acceleration, m/s^2
N	Number of houses served.

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1. INTRODUCTION

As research into the characteristics of wastewater has become more extensive, and as the potential health and environmental effects have become more comprehensive, the body of scientific knowledge has expanded significantly (Metcalf and Eddy, 2003). Most of the new technologies are developed and designed to deal with health and environmental concerns. Specially, in third world countries, the unplanned urban development results in divesting wastewater collecting and treating problems.

In such countries, wastewater is usually directed to near by rivers, or via the pits into the groundwater. Both of these solutions can be treated as potential health and environment polluters. The search for pilot solution is generally preferred due to the lack of financial help.

All these concerns finally motivate the experts to minimize the cost of wastewater network system. During 1980's, the years of the international drinking water supply and sanitation decade, much emphasis is placed by the international agencies on the promotion of on-site sanitation system, though some development work was done on settled sewerage, (Otis and Mara, 1986) and alternative sewer system (WPCF, 1986).

Among the above, simplified sewerage were only implemented by researchers from Brazil and Pakistan (Rodriqves de Meio, 1985; de Andrade Neto, 1985; Azevedo Netto, 1992) and (Sinnatamby, Mara and Mcgarry, 1986). The reason is clearly due to the financial problems in there countries.

As it is mentioned in Mara and Guimares, 1999, it is now abundantly clear that simplified sewerage is generally the sanitation technology of first choice in high-density low-income urban and semi-urban areas.

Advantages:

- A small diameter pipe at shallow depths reduces cost over conventional sewerage systems.
- Utility maintains main sewer pipes only and community maintains local network.
- High service level, suitable for high density communities.

Disadvantages:

- Cost remains a problem for low-income communities.
- Reliability of water services is essential.
- Problems of downstream waste management remain for the utility.

Figure 1.1, which shows that, as the population density increases, simplified sewerage can become cheaper than on-site sanitation systems.

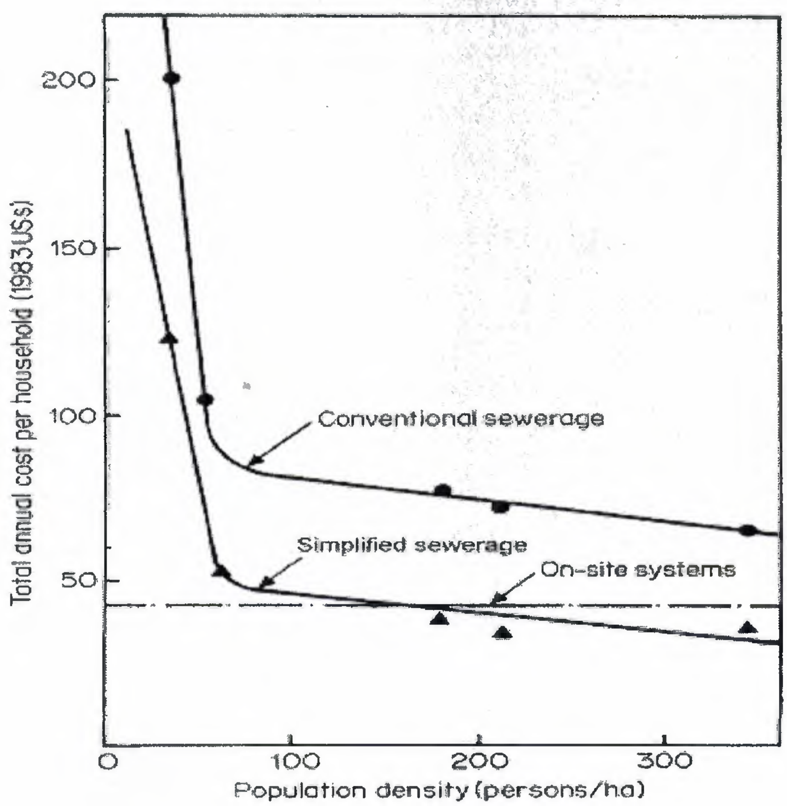

Figure 1.1 Costs of conventional and simplified (condominial in-block) sewerage, and on-site sanitation in Natal in northeast Brazil in 1983.

Simplified sewerage is an off-site sanitation technology that removes all wastewater from the household environment. Conceptually it is the same as conventional sewerage, but with conscious efforts made to eliminate unnecessarily conservative design features and to match design standards to the local situation.

In this study, the fundamental theory of simplified sewer design is discussed by the help of readily available computer program called, "Pc-based simplified sewer design". It is used to analyze the effect of the sewer diameters and gradients for different cases, and finally, a case study is carried over to apply the simplified sewerage techniques for peri-urban Taskinkoy region.

2. HYDRAULIC DESIGN OF SIMPLIFIED SEWERAGE

2.1 INTRODUCTION

The fundamental of theory of simplified sewer design is to be discussed in this chapter. Firstly, the peak daily wastewater flow in the length of sewer being designed is described which is directly related with the population served by length of sewer under consideration. Since the usual pipe shapes used for sewer system is circular, the trigonometric properties of a circular section is investigated and the hydraulic section characteristics are analyzed. The velocity of flow which in term determines the rate of flow is defined by Gauckler-Manning equation. The equation is a function of sewer gradient and hydraulic radius. Tractive tension is explained in detail, and the minimum sewer gradient based on the design minimum tractive tension is explained in detail. The procedure for calculating the sewer diameter for determining the maximum number of houses served by a sewer of given diameter is also discussed. Finally, the results of a simplified sewer design are presented, with a comparison of designs based on the Gauckler- Manning, Colebrook-White and Escritt equations. The further investigation on this chapter can also be obtained from Mara (1996) (Yao, 1974; Machado Neto and Tsutiya, 1985; de Melo, 1985 and 1994; Bakalian *et al.*, 1994).

2.2 WASTEWATER FLOW

It is necessary to estimate the daily peak flow capacity available in order to be able to design a suitable sewerage network system. This can be achieved by the use of the following equation:

$$q = k_1 k_2 pw / 86400 \qquad (2.\ 1)$$

Where q = daily peak flow rate, l/s

K_1 = peak factor

K_2 = return factor

p = population served by length of sewer under consideration

w = average water consumption, l/ capita / day

and 86 400 is the number of seconds in a day. Peak factor, K_1 is the ratio between the daily peak flow and average daily flow. A suitable design value for peak flow for simplified sewerage is 1.8. Return factor, K_2 however, is the ratio between rate of wastewater flow and water consumption. Usually the ratio is 0.85. In accordance with the values given for K_1 and K_2 equation (2.1) can be simplified to:

$$q = 1.8 \times 10^{-5} pw \qquad (2.2)$$

The design values given above for the peak flow factor, K1 and the return factor, K2 (1.8 and 0.85 respectively) have been found to be suitable in Brazil, but they may need changing to suit conditions elsewhere – especially if stormwater (for example, roof drainage water) is discharged into the simplified sewer. However, this should not be permitted to occur as the resulting design for what is in practice partially combined sewerage system would be based on a much higher value for K_1 (perhaps as high as 3 or 4).

Variations in the value of K_2 have a much lower impact on design, except in middle and high-income areas where a large proportion of water consumption is used for lawn-watering and car-washing. Thus, the return factor can take a value of 0.65 or 0.95 depending on the social and cultural environment of the population which is served. it is clear that as the water consumption increases return factor will decrease.

2.2.1 Minimum daily peak flow

The simplified sewer design equation 2.1 or 2.2 is used to calculate the daily peak flow in the length of sewer under consideration. The design equation however is subject to a minimum value which is limited by 1.5 l/s. This minimum flow is not justifiable in theory but, as it is approximately equal to the peak flow resulting from flushing a WC, it gives sensible results in practice, and it is the value recommended in the current sewer design codes (ABNT, 1986; Sinnatamby, 1986). With the use of this minimum value for the peak daily flow, the values used for K_1 and K_2 in equation 2.1 become less important, especially for short lengths of sewer. For example, for a length of sewer serving 500 people with a water consumption of 80 litres per person per day and using a return factor of 0.85, the average daily wastewater flow can be given as following:

$$q = k_2 pw / 86400 \tag{2. 3}$$

$$= 0.85 \times 500 \times 80 / 86400$$

$$= 0.4 \text{ l/s}$$

For the minimum peak daily flow of 1.5 l/s, this is equivalent to a K_1 value of (1.5/0.4) = 3.75. Thus for condominial sewers serving even quite a large number of people, there is an inherent allowance for at least some stormwater.

2.3 PROPERTIES OF A CIRCULAR SECTION

The flow in simplified sewers is always following a hydraulic flow, resembling the 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 2.1. 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 is the area of flow divided by the wetted perimeter. The wetted perimeter is the surface of the pipe which is in contact with wastewater. The breadth of flow is used for the calculation of the risk of hydrogen sulphide generation.

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

(5) the angle of flow (θ), expressed in *radians*;

(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.

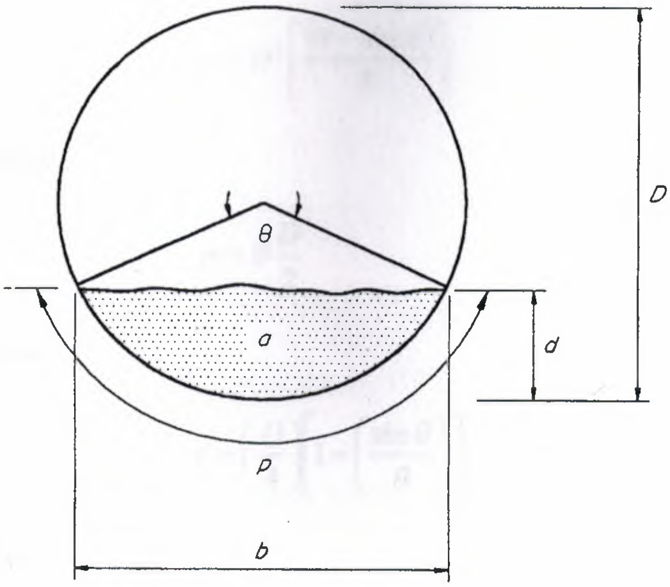

Figure 2.1 Definition of parameters for open channel flow in a circular sewer, Mara (1996).

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

$$0.2 < \frac{d}{D} < 0.8$$

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 relationship between the above parameter, can be defined in terms of each other, as given in the following equations:

(a) Angle of flow:

$$\theta = 2\cos^{-2}\left[1 - 2\left(\frac{d}{D}\right)\right] \quad (2.4)$$

(b) Area of flow:

$$a = D^2\left[\frac{(\theta - \sin\theta)}{8}\right] \tag{2.5}$$

(c) Wetted perimeter:

$$p = \theta\frac{D}{2} \tag{2.6}$$

(d) Hydraulic radius:

$$r = \left(\frac{D}{4}\right)\left[1 - \left(\frac{\sin\theta}{\theta}\right)\right] \tag{2.7}$$

(e) Breadth of flow:

$$b = D\sin\left(\frac{\theta}{2}\right) \tag{2.8}$$

When d = D (that is, when the sewer is flowing just flow), then a = A = $\frac{\pi D^2}{4}$;

p = P = πD and r = R = $\frac{D}{4}$.

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

$$a = k_a D^2 \tag{2.9}$$

$$r = k_r D \tag{2.10}$$

The coefficients k_a and k_r are given from equations 2.5 and 2.6 as:

$$k_a = \frac{1}{8}(\theta - \sin\theta) \tag{2.11}$$

$$k_r = \frac{1}{4}\left[1 - \left(\frac{\sin\theta}{\theta}\right)\right] \tag{2.12}$$

When $a = A$ and $r = R$, then $k_a = \frac{\pi}{4}$ and $k_r = 0.25$.

2.3.1 Hydrogen Sulphide Generation

Hydrogen sulphide (H_2S) generation in sewers leads to microbial corrosion of the crown of concrete and asbestos – cement sewers (Figure 2.2). The likelihood of H_2S generation is given by Pomeroy's (1990) in term of z factor his definition is given as follows:

$$z = 3(BOD_5)(1.07)^{T-20} i^{-1/2} q^{-1/3} \left(\frac{p}{b}\right) \tag{2.13}$$

Where BOD_5 = 5-day, 20 ^{o}C biochemical oxygen demand of the wastewater (mg/l)

T = temperature, ^{o}C

i = sewer gradient, m/m

q = wastewater flow, l/s

p = wetted perimeter, m

b = breadth of flow (see Figure 2.1), m

and 3 is the conversion factor resulting from changing the units of q from ft^3/s in Pomeroy's original equation to l/s.

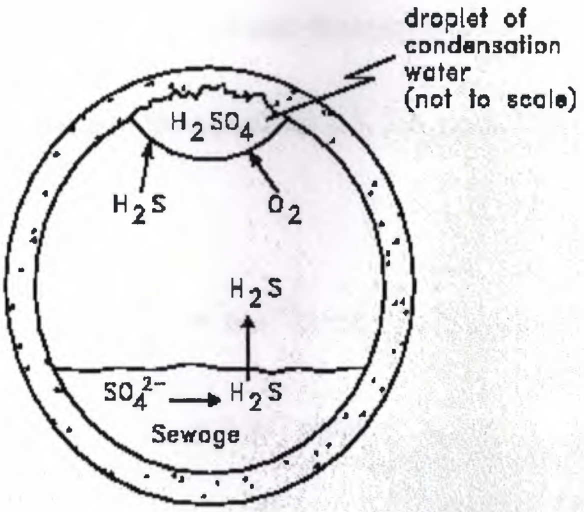

Figure 2.2 Microbially induced corrosion of the crown of concrete or asbestos cement sewers.

Sulphates in the wastewater are reduced anaerobically by sulphate-reducing bacteria to hydrogen sulphide, some of which leaves the wastewater to raise its partial pressure in the atmosphere above the flow (Henry's law), and then some of this H_2O goes into solution (Henry's law again) in droplets of condensation water clinging to the sewer crown – this H_2S is oxidized by the aerobic bacterium *Thiobacillus thioparus* to sulphuric acid (H_2SO_4), which corrodes the concrete. Sewer crown collapse within 10-20 years is common.

The value of Z calculated from Equation 2.13 is used diagnostically as follows:

$Z < 5000$: H_2S generation unlikely

$5000 < Z < 10\,000$: H_2S generation possible

$Z > 10\,000$: H_2S generation very likely

With simplified sewerage, hydrogen sulphide generation can be expected to be a common problem. For example, for a flow of 1.5 l/s of wastewater with a BOD_5 of 250 mg/l at 25^{O}C in a sewer laid at 1 in 214 and flowing at a proportional depth of flow of 0.2, Z can be calculated through the equations 2.4, 2.6 and 2.8 calculation of $\frac{p}{b}$ for $\frac{d}{D}$ = 0.2 can be given as:

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

$$= 0.927 \text{ radian}$$

$$\frac{p}{b} = \frac{\left(\frac{\theta}{2}\right)}{\sin\left(\frac{\theta}{2}\right)}$$

$$= 1.159$$

$$z = 3 \times 250(1.07)^5\left(\frac{1}{214}\right)^{-1/2}(1.5)^{-1/3}(1.159)$$

$$= 16\,000$$

Thus H_2S generation is very likely, and this is why the small diameter pipes used in simplified sewerage schemes should normally be of either vitrified clay or PVC.

2.4 VELOCITY OF FLOW EQUATION

In the 18th and 19th centuries three principal equations for the velocity of flow in open channels and pipes were developed. These are:

(1) The Chézy equation,

(2) The Gauckler-Manning equation, and

(3) The Darcy-Weisbach equation.

(4) The Escritt equation.

The Chézy and Gauckler-Manning equations are related as the Ganguillet-Kutter equation for the Chézy coefficient of flow resistance includes the Kutter roughness coefficient, n which is identical to that used in the Gauckler-Manning equation.

The Darcy-Weisbach equation introduces the Darcy-Weisbach friction factor, f, which for turbulent flow in both rough and smooth pipes is given by the Colebrook-White equation used in modern sewer design (see, for example, Butler and Pinkerton, 1987). The discussion that follows is based principally on Chow (1959), Yen (1992) and Chanson (1999).

2.4.1. Gauckler-Manning Equation

In 1889 Robert Manning improved the chezy formula by relating the velocity of flow in a sewer to the sewer gradient and the hydraulic radius (Manning, 1890). The formula is commonly known as the Manning equation. However, as pointed out by Williams (1970) and Chanson (1999), it should be known as the Gauckler-Manning equation since Philippe Gauckler published the same equation 22 years earlier (Gauckler, 1867 and 1868). The Gauckler-Manning equation is:

$$v = \left(\frac{1}{n}\right) r^{2/3} i^{1/2} \qquad (2.14)$$

where v = velocity of flow at *d/D*, m/s

n = dimensionless Ganguillet-Kutter roughness coefficient.

r = hydraulic radius at *d/D*, m

i = dimensionless sewer gradient, m/m

Applying the rules of continuity results in;

$$q = \left(\frac{1}{n}\right) a r^{2/3} i^{1/2} \quad (2.15)$$

Where q = flow in sewer at *d*/*D*, m^3/s

Using equations 2.9 and 2.10, equation 2.15 becomes:

$$q = \left(\frac{1}{n}\right) k_a D^2 (K_r D)^{2/3} i^{1/2} \quad (2.16)$$

The usual design value of the Ganguillet-Kutter roughness coefficient, n is 0.013. This value is used for any relatively smooth sewer pipe material (concrete, PVC or vitrified clay) as it depends not so much on the roughness of the material itself, but on the roughness of the bacterial slime layer which grows on the sewer wall.

2.4.2 The Escritt Equation

Escritt (1984) gives his equation for wastewater flow in circular sewers in the form :

$$v = 26.738\, D^{0.62}\, i^{1/2} \quad (2.17)$$

Where v = velocity of flow, m/min

D = diameter, *mm*

Changing the units of v to m/s and D to m and writing D as 4r gives:

$$v = \left(\frac{1}{0.013}\right) r^{0.62} i^{1/2} \quad (2.18)$$

The hydraulic radius, r in this equation is "not the cross-sectional area divided by the wetted perimeter, but averaged, with remarkable accuracy, the cross-sectional area divided by the sum of the wetted perimeter and one-half the width of the water-to-air surface" (Escritt, 1984), that is:

$$r = \frac{a}{\left[p + \left(\frac{b}{2}\right)\right]} \qquad (2.19)$$

Equation 2.18 shows the Escritt equation to be a variant of the Gauckler-Manning equation, with n taken as 0.013 for slimed sewers, and with r defined by equation 2.19 and having the exponent 0.62 rather than 2/3.

2.5 TRACTIVE TENSION

Tractive tension (or boundary shear stress) is the tangential force exerted by the flow of wastewater per unit wetted boundary area. It is denoted by the symbol τ and has units of N/m^2 (i.e. Pascals, Pa). As shown in Figure 2.3, and considering a mass of wastewater of length l and cross-sectional area *a*, which has a wetted perimeter of p, the tractive tension is given by the component of the weight (*W*, Newtons) of this mass of wastewater in the direction of flow divided by its corresponding wetted boundary area (i.e. the area in which it is in contact with the sewer = *pl*):

$$\tau = \frac{w \sin\phi}{pl} \qquad (2.20)$$

The weight W can be expressed in terms of specific weight and the volume of the sewer. Gives as:

$$w = \rho g a l \qquad (2.21)$$

Where ρ = density of wastewater, kg/m^3

g = gravitational acceleration, m/s^2

So that, since *a*/p is the hydraulic radius, *r*:

$$\tau = \rho g r \sin\phi \qquad (2.22)$$

For $d/D = 0.2$, the minimum value used in simplified sewerage – that is, from equations 2.4, 2.11 and 2.12, for $k_a = 0.1118$ and $k_r = 0.1206$; and with n = 0.013, r = 1000 kg/m^3 and $g = 9.81$ m/s^2, equation 2.26 becomes:

$$I_{\min} = 2.33 \times 10^{-4} (\tau_{\min})^{16/13} q^{-6/13} \quad (2.27)$$

A good design value for $\tau_{\min}$ in simplified sewerage is 1 Pa; thus:

$$I_{\min} = 2.33 \times 10^{-4} q^{-6/13} \quad (2.28)$$

In this equation the units of q are m^3/s. Changing them to litres/second gives:

$$I_{\min} = 5.64 \times 10^{-3} q^{-6/13} \quad (2.29)$$

Equations 2.28 and 2.29 are for a value of $\tau_{\min}$ of 1 Pa. Yao (1974) recommends values of $\tau_{\min}$ for sanitary sewers of 1-2 Pa, and 3-4 Pa for stormwater or combined sewers. Designers must make an appropriate choice for $\tau_{\min}$, and use equation 2.27 for values > 1 Pa. Values of $\tau_{\min}$ > 1 Pa have a large influence on the value of $I_{\min}$. For example, for a flow of 1.5 l/s, equation 2.27 gives:

$\tau_{\min}$ (pa)	$I_{\min}$
1	1 in 213
1.5	1 in 130
2	1 in 91

In low-income areas $\tau_{\min}$ is usually taken as 1 pa and the minimum of $I_{\min}$ is taken as 0.5% this has not resulted in any significant operational problems (Luduvice, 2000).

2.7 SEWER DIAMETER

Equation 2.16 can be rearranged, as follows, writing i = $I_{\min}$:

$$D = n^{3/8} k_a^{-3/8} k_r^{-1/4} \left(\frac{q}{I_{\min}^{1/2}} \right)^{3/8} \qquad (2.230)$$

In this equation the units of D are m, and the units of q are m^3/s.

The sewer diameter is determined by the following sequence of calculations:

(1) Calculate using equation 2.2, the initial and final wastewater flows (q_i and q_f, respectively, in l/s), which are the flows occurring at the start and end of the design period. (The increase in flow is due either to an increase in population or an increase in water consumption, or both.)

If the flow so calculated is less than the minimum peak daily flow of1.5 l/s, then use in (2) a value of 1.5 l/s forq_i.

(2) Calculate $I_{\min}$ from equation 2.29 with $q = q_i$.

(3) Calculate D from Equation 2.30 using q = q_f (in m^3/s), again subject to a minimum value of 0.0015 m^3/s, for d/D = 0.8 (i.e. for k_a= 0.6736 and k_r= 0.3042 from equations 2.4, 2.11 and 2.12).

In this design procedure, the value of q_i is used to determine $I_{\min}$ and the value of q_f is used to determine D. The diameter so calculated is unlikely to be a commercially available size, and therefore the next larger diameter that is available is chosen (i.e. if D = 86 mm, say, then choose 100 mm).

The minimum advisable diameter used in simplified sewerage is 100 mm.

2.8 NUMBER OF HOUSES SERVED

In the detailed design of condominial sewers it is useful to know the maximum number of houses that can be served by a sewer of given diameter. The procedure for calculating this is shown here – *as an example only* – for a household size of 5, a per capita water consumption of 100 l/d, a peak factor of 1.8 and a return factor of 0.85, the peak flow per household (q, l/s) is given by equation 2.2 as:

$$q_h = 1.8 \times 10^{-5} \rho w$$

$$= 1.8 \times 10^{-5} \times 5 \times 100$$

$$= 0.009 \text{ l/s per household.}$$

If it is assumed that the housing area is fully developed (i.e. that there is no space for further houses), then any increase in wastewater flow will be due to an increase in water consumption.

Designing the sewer for an initial d/D of 0.6, allows for an increase in water consumption to just under 150 liters per capita per day when d/D will be the maximum value of 0.8 (Mara, 1996). Equations 2.16 (with i = $I_{\min}$) and 2.26 are now solved for d/D = 0.6 (i.e. for k_a = 0.4920 and k_r = 0.2776), with $\tau_{\min}$ = 1 Pa and with q in l/s, as follows:

$$I_{\min} = 0.00518 q^{-6/13} \qquad (2.31)$$

$$D = 0.0264 \left(\frac{q}{I_{\min}^{1/2}} \right)^{3/8} \qquad (2.32)$$

Thus, with D in mm:

$$q = 9.8 \times 10^{-5} D^{13/6} \qquad (2.33)$$

The peak flow per household is 0.009 l/s, so that q is given by:

$$q = 0.009 N \qquad (2.34)$$

where N = number of houses served. Thus:

$$N = 10.89 \times 10^{-3} D^{13/6} \tag{2. 35}$$

Equation 2.31 shows that, for the design values assumed, a 100 mm diameter sewer can serve up to 234 houses. For any other set of design parameters (including the initial value of *d/D*) an equation corresponding to equation 2.35 has to be derived in the manner shown above.

2.9. DESIGN COMPARISONS

In Sections 2.3 – 2.6 the Gauckler-Manning equation was used to exemplify the basis of the hydraulic design of simplified sewers. Although it is the only equation to have been used to date for simplified sewer design in practice, there are two other principal equations which are currently used for the hydraulic design of conventional sewers, and which could in principle therefore be used for simplified sewer design. They are:

(1) the Colebrook-White equation (Colebrook, 1938; see also Butler and Pinkerton, 1987 and HR Wallingford and Barr, 1994), and

(2) the Escritt equation (Escritt, 1984).

3. THE PLANNING AND DESIGN PROCESS

3.1 INTRODUCTION

The theory introduced in chapter 2 allows a sewer system to be analyzed such that, sewer diameters and gradients can be determined. This step only covers the first step of the overall planning and design process. In this chapter of a sewer system, sewer design process is explained and the related pc-based program is presented. This chapter is subdivided as follows:

Section 3.2 is concerned with the initial assessment of sanitation options. The assessment of technical options is explained and the issues relating to the management options for simplified sewerage are explored. Section 3.3 sets out the sewerage planning process, from the decision to adopt simplified sewerage to the development of the overall sewerage layout. It explains what information is needed for the planning process and explores the factors that will influence the area to be included in a sewerage scheme. This leads in to the development of a draft sewerage plan. In most cases, it will then be necessary to carry out physical and social surveys before finalising sewer routes. Planning leads into detailed design. Section 3.4 considers various aspects of detailed design, including the selection of design parameters (input parameters, those that override design calculations, and output parameters), and the design of condominial sewers and public collector sewers.

3.2 INITIAL ASSESSMENT OF SANITATION OPTIONS

Two basic questions should be asked at the beginning of the planning process.

These are:

· What sanitation options are feasible in the local situation? and

· Assuming that simplified sewerage is feasible, what arrangements are possible for managing the construction and subsequent operation and maintenance of the local condominial systems? Each of these questions is considered below.

3.2.1 Technical options

This is the stage at which the decision to use simplified sewerage will be made. Simplified sewerage should only be considered where a reliable water supply is or can be made available on or near each plot so that total water use is at least 60 liters per person per day. Where this basic criterion cannot be met, other options should be evaluated. Sewers, preceded by settlement tanks and carrying 'settled' wastewater might be considered when water use is lower, perhaps down to 30 liters per person per day. Settled sewerage (also called small-bore, or solids-free, sewerage) is described by Otis and Mara (1985) and Mara (1996).

Other factors to be considered are population density, the arrangements for effluent disposal and the preferences of the local people; for evaluating on-site sanitations options the plot size, the infiltration capacity of the soil and the potential for groundwater pollution should also be considered (see Franceys *et al.*, 1992; Cotton and Saywell, 1998; and GHK Research and Training, 2000). Simplified sewerage became cheaper than on-site systems at a population density of around 160 people per hectare. While the precise figures were particular to northeast Brazil at that time, the broad pattern may be expected to occur elsewhere. Simplified sewerage should always be considered as an option when population densities exceed about 150 people per hectare.

When comparing costs between different sanitation technologies, the following points must be taken into account: (Mara, D., 2001)

• The cost of sewerage is not confined to the cost of local sewers. The cost of any collector and trunk sewers and that of treatment have also to be included.

• Most on-plot sanitation systems do not cater for sullage (i.e. the wastewater from sinks, showers etc.). It may be necessary to include separate drainage facilities for sullage and this cost has to be taken into account in any cost comparison.

Simplified sewerage is more likely to be viable where an existing collector sewer with spare capacity is available reasonably close at hand. The existing sewer represents a sunk cost and the cost of simplified sewerage is therefore reduced.

In theory, the cost of sewered sanitation can be reduced by treating wastewater locally, thus removing the need for expensive trunk mains. In practice, lack of both land and the skills necessary to operate local treatment facilities may prevent the adoption of this option.

The operating costs of the various sanitation systems need to be considered when choosing an appropriate technology. For sewerage, the cost of any pumping that may be required must be considered, together with who is going to pay for it. The cost (and availability and reliability) of WC flushing water also needs to be included.
User preferences are likely to influence choice when there is little to choose between two sanitation technologies. In general, users prefer sewers because they remove all

wastewater (i.e. both toilet wastewater and sullage) from the house and, if properly constructed, they require relatively little maintenance. In some cases, local people may be opposed to sewers because of previous bad experiences. These normally relate to bad design, bad operation and maintenance, misuse (for instance dumping solid waste in the sewers) or some combination of the three. In such circumstances, the reasons for the previous problems should be ascertained and the ways in which they can be overcome should be discussed with the users.

3.2.2 Management options

It is important to consider the possible management options for any proposed sanitation system from the very beginning of the planning process. In general, the more small-scale and local a sanitation system, the better the prospects for local management. So, it would appear that on-plot sanitation systems such as pit latrines and pour-flush toilets discharging to leach pits can be managed by individual householders, while city-wide sewage disposal systems must be managed at the municipal level. In practice, household sanitation facilities, sewers and wastewater disposal facilities together form a hierarchical wastewater disposal system, as shown diagrammatically in Figure 3.1.

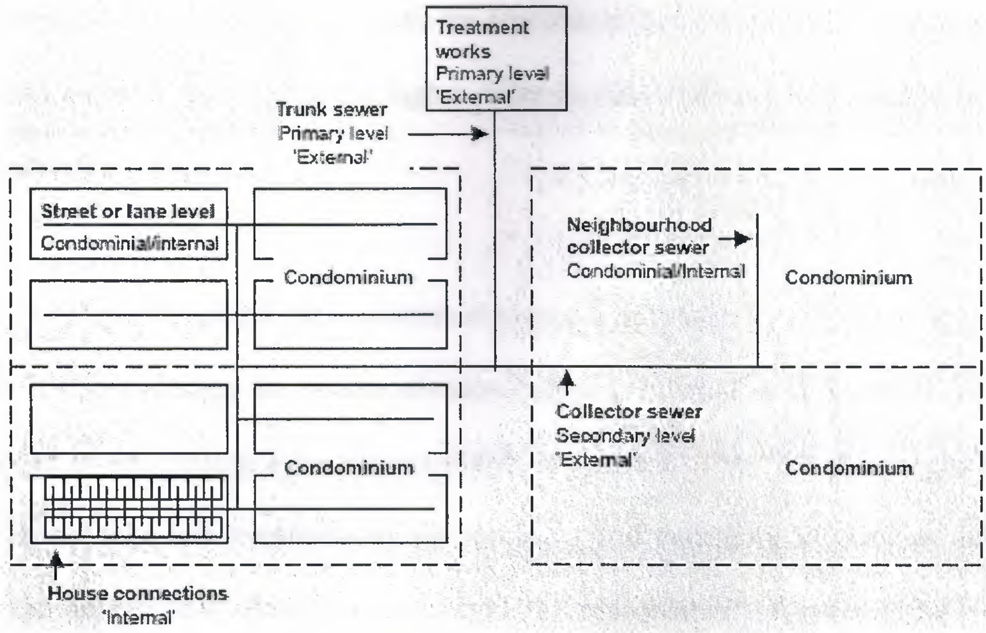

Figure 3.1 Sewerage as a hierarchial system.

Figure 3.1 suggests that a second division is possible, between those system components that serve particular areas or 'condominiums' and those that have a wider city or city district function. A condominium will normally include a number of streets or lanes that can be sewered to one connection with a higher-order collector sewer. The condominial systems do not have to be managed by the same organization that manages the higher-order facilities and may be suitable for management by a local organization, either the local community itself or a contracted private sector organization. In the latter case, the contract should ensure that the contractor is responsible to the local community for the performance of the system.

This division of responsibilities can result in better management of local facilities because it ensures that responsibility for the local facilities lies those (the community members) who are directly affected by the performance of these facilities. At the same time, it ensures that organizations such as municipalities, specialist sewerage agencies and government departments can make the best use of their resources by focusing on the operation and maintenance of the higher-order facilities that are not suitable for local management.

This is the thinking behind the condominial approach as originally developed in Brazil. It also underlies the similar division between 'internal' and 'external' facilities developed by the Orangi Pilot Project (OPP) in Pakistan. The OPP philosophy is that users should take full responsibility for providing and managing all internal facilities, while the government should similarly take full responsibility for managing external facilities, including collector and trunk sewers and wastewater treatment facilities. The exact details of the division of responsibilities should be decided in the light of the local situation under consideration.

Local management does not mean that all the tasks associated with operating and maintaining sewers have to be carried out by users themselves. Management options for operation and maintenance are extremely important in ensuring system sustainability.

It is extremely important to evaluate what management arrangements are possible in the local situation. In particular, community management should not be considered an option for a local simplified sewerage schemes connected to a municipal system when the operators of the municipal system do not recognize the right of local users to manage their own system.

3.3 PLANNING FOR SEWERAGE

In this section, the steps that lead from the decision to adopt simplified sewerage to the development of a sewer layout that can be analyzed using the PC-based sewer design program is described. These steps can be summarized as follows:

(1) Collect existing information, focusing particularly on maps and plans of the area to be sewered and adjacent areas,

(2) Determine the area to be included in the sewerage plan, based on topography, the location of existing sewers and the limits of existing and future development,

(3) Develop a draft sewerage plan, showing the routes of the main collector sewers and the approximate areas of the various condominial systems,

(4) Undertake additional surveys as required to allow sewer routes and the areas of condominial systems to be confirmed, so that detailed design can be carried out, and

(5) Finalize the overall sewerage plan and plot the sewer routes at an appropriate scale or scales.

3.3.1 Collection of existing information

The first task in the planning process is to collect all available information on the area to be sewered. In particular, existing topographical maps and any maps showing the routes of existing drains and sewers should be collected, as these are needed to define the area to be sewered and determine the overall sewer layout. This information may be available on a number of maps and plans; if this is the case, as much information as possible should be transferred to one base plan.

Information on existing management arrangements and responsibilities also needs to be collected. This provides a sound basis for developing institutional arrangements to manage the proposed system. One of the advantages of dividing sewerage schemes into condominial and collector systems lies in the possibilities for local management of the former. With this in mind, information on existing community structures and systems should be collected, so that the potential for local management of condominial systems can be assessed.

3.3.2 Area to be included

The next task is to decide the area to be included in the scheme. There are two possible situations. The first is that the design is for an exclusively local system, which can be connected to a local treatment facility or an existing collector sewer. The second is that there is a need to look at the sewerage needs of a wider area, including both local condominial sewers and public collector sewers.

In the first case, the decision on the area to be included in the scheme is relatively straightforward. In general, its boundaries will coincide with those of the existing or planned housing scheme that is to be sewered. The main task will be to determine the routes of the internal condominial sewers and the points at which they will discharge to a treatment site or existing sewer.

The second situation is more complicated in that the boundaries of the area to be drained by the collector sewers may not be immediately obvious. The important point is to ensure that the overall situation is taken into account, as defined by natural drainage areas, the location of existing sewers and possible treatment/disposal locations. The boundaries of natural drainage areas should be fairly obvious in hilly or undulating areas. They may be much less obvious where the topography is flat. Where this is the case, the routes of existing natural watercourses, drains and sewers will give a good idea of existing drainage patterns. By plotting existing drains on a suitable plan the approximate boundaries of drainage areas and the main drainage paths should be able to be defined. As this 'context plan' is developed, any land that might be available for local treatment should be identified. This allows the relationship between the scheme area and possible treatment/disposal facilities and sites to be explored. This in turn enables the possible advantages of enlarging the scheme to cover surrounding areas to be assessed. (Mara, D., 2001).

3.3.3 Development of a draft sewerage plan

It should now be possible to develop a draft sewerage plan. Whether this covers a local system or the sewerage needs of a wider area, the same basic principles apply. Sewers should be routed as close as possible to natural drainage routes, while taking into account existing land development and ownership patterns. In general, collector sewers should be routed in public rights of way which are close as possible to natural drainage routes. Where an existing drainage channel is located along a narrow right of way between existing houses, the sewer should preferably be rerouted along adjacent roads where there is better access for maintenance.

The first step is to decide the routes of the main public collector sewers and then consider how local condominial systems can be joined to them. In general, public collector sewers should be designed to include flows from all parts of the drainage area that are or are likely to be sewered. Failure to do this will mean that the sewers will be undersized, if not immediately then certainly in the future.

Once the routes of the main public collector sewers are decided, preliminary proposals can be made for the routes of condominial systems. It is possible that as this is done, minor adjustments to the routes of the main sewers may need to be made.

Figure 3.2 shows a possible sewer layout for an area including a single public collector sewer and a number of condominial sewers. Note that the main collector sewer is routed along roads, keeping as close as possible to the natural drainage route that can be determined by the contours. Some of the condominial systems connecting to the main sewer are routed along roads, while those at the top of the figure are assumed to be in-block systems, passing through the private space between houses.

The accuracy with which sewer layouts can be plotted at this stage will depend on the accuracy of the available plans and the availability of information on ground levels. Final decisions on the limits of condominial systems may also be influenced by social factors. The next section considers the steps to be taken to collect and record the physical and social information necessary for detailed design.

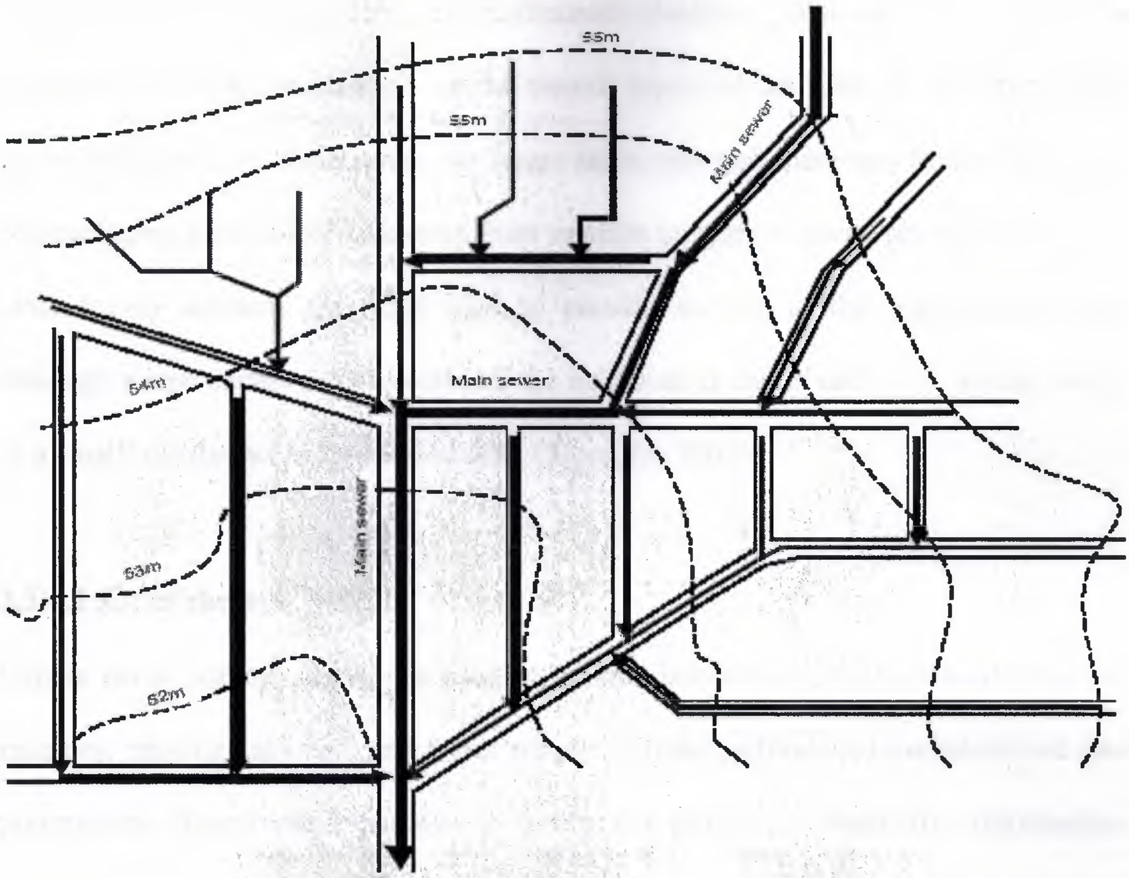

Figure 3.2 Sewer plan should respect the natural topography.

3.3.4 Physical and social surveys

If accurate survey information is not available, detailed physical and social surveys are generally required. Each is briefly considered in turn below.

3.3.4.1 Physical surveys

Physical surveys are required in order to determine sewer routes and levels. If existing plans exist, it may be possible to use them, at least for preliminary design. However, checks on their accuracy should always be made, and they must be updated to include any developments that have taken place since they were produced.

Where plans are non-existent or insufficiently detailed, additional surveys will be required to provide information on the overall layout of the area. A full triangulated survey will normally be necessary for larger areas, although there may be the possibility of developing a municipal base-map from satellite imagery or aerial photographs. Plane table survey methods are often used to provide surveys at the condominial level, although a tape survey may provide all the information that is necessary for the design of a small, relatively uncomplicated area. (Mara, D., 2001).

3.3.4.2 Social surveys

Simple social surveys should be used to provide information on household sizes and incomes, existing sanitation and water supply facilities, attitudes to sanitation and user preferences. Questionnaire surveys are useful for providing quantitative information. Semi-structured interviews and focused group discussions are more likely to provide information on attitudes and preferences.

The options for management can be explored in community meetings, although it will be wise to back these up with smaller meetings with particular groups. This is because minority viewpoints may not emerge in open community meetings.

3.3.5 Final sewer routes

Once good survey information has been obtained, it can be recorded on suitable plans and detailed design of the system can commence. Minor changes to the routes of collector sewers may be required as a result of improved survey information. More substantive changes may be necessary in condominial systems as a result of the findings of both the physical and social surveys.

The preferred options for condominial sewers should be decided in consultation with local people, bearing in mind the management arrangements to be adopted.

3.4 DETAILED DESIGN

3.4.1 Introduction to the design process

Detailed design requires a combination of hydraulic calculations and the application of standard designs, procedures and details. In some cases, for instance the minimum allowable sewer diameter, the application of a design standard may override the results of design calculations.

Sections 3.4.2 to 3.4.5 are concerned with design parameters. The way in which they can be categorized is explained first in Section 3.4.2, and then input parameters, parameters that over-ride design calculations and output parameters are discussed in Sections 3.4.3 – 3.4.5.

Attention then turns to the design calculations. It is possible to carry out these for sewer systems as a whole. Alternatively, it is possible to design individual condominial systems first and then to input some of the data from these calculations into the calculations for the design of public collector sewers. The most appropriate approach will depend on the designer's preferences and the local situation. The design of a local condominial system is considered first in Section 3.4.6, and the design of public collector systems in Section 3.4.7.

3.4.2 Categories of design parameter

Design parameters include those that are required for calculation purposes and those that over-ride design calculations. The former include the average household size, the average per capita water consumption, the return factor and the various factors that affect the total design flow. These are introduced in Section 3.4.3. Parameters that over-ride design calculations are the minimum sewer diameter and the minimum design flow, and these are considered in Section 3.4.4. There is only one design output parameter and this is the minimum sewer gradient which is considered in Section 3.4.5.

There is a further category of design parameters which emerge from investigations of field conditions. These include the type of access allowable, the Manhole/chamber spacing and the minimum allowable chamber dimensions.

3.4.3 Design input parameters

3.4.3.1 Average household size:

This is multiplied by the number of houses in an area or along a sewer leg to determine the design population in that area or contributing to the sewer leg. Results from the social survey will provide information on the average household size in the area to be sewered.

3.4.3.2 Average per capital water consumption:

This is multiplied by the design population for any area or sewer leg to calculate the total amount of water used during a typical day. Information on average per capita water consumption may be available from meter readings. Failing this, the local water authority may keep records of average per capita consumption in different areas and types of development. The likely per capita water consumption at both the beginning and the end of the design period (which will typically be 30 years) has to be considered.

3.4.3.3 Return factor:

This defines the percentage of total water consumption that will be discharged to the sewer. It is often assumed to be 80% or 85%, although there are indications that lower return factors may be appropriate in some areas. The wastewater flow from an area will be equal to the water consumption in the area multiplied by the return factor.

3.4.3.4 Peak wastewater flow factor:

This is required to allow for the fact that the wastewater flow varies through the day, reaching a peak when people get up in the morning and falling to almost nothing during the night. The peak *foul* flow in any sewer can be taken as the average flow in that sewer multiplied by the peak factor. Peak factors tend to decrease as the population contributing to the flow increases. However, even for a population of a few hundred, the peak factor is unlikely to exceed 2. (Mara, D., 2001).

3.4.3.5 Groundwater infiltration:

This needs to be considered where some sewers are laid below the groundwater table. Infiltration is commonly estimated on the basis that it is a set percentage of the average per-caput wastewater flow. A theoretically more accurate approach will be to assume an infiltration rate per unit length of sewer. The first method is simpler. Furthermore the accuracy of available information will normally be insufficient to justify the adoption of the second approach. However, laying sewers below the groundwater table should be avoided wherever possible.

3.4.3.6 Allowance for stormwater:

Sewers can be designed as separate, partially combined or combined. Separate sewers carry only wastewater; partially combined sewers are designed to carry some stormwater in addition to wastewater, while combined sewers are designed to carry the full wastewater and stormwater flows. Combined sewerage has several disadvantages. In all but the driest climates, the size of sewer required to carry the full stormwater run-off is likely to be much larger than that required for the wastewater flow. Combined sewerage thus requires a high level of investment, which is not utilized except in wet weather. Combined sewers also have the disadvantage that stormwater run-off often carries a high concentration of grit and other suspended solids and this can lead to higher rates of silting. Sewers have therefore to be laid at greater gradients than would be required if they carried only wastewater. For these reasons, simplified sewer systems should not be designed as combined.

Normal practice in many industrialized countries is to provide nominally separate wastewater and stormwater systems. However, in practice, it is extremely difficult to exclude all storm flows and so separate systems are always designed with some allowance for the entry of storm flows. As already indicated, the peak wastewater flow will not exceed twice the average dry weather wastewater flow.

The situation in low-income periurban settlements in developing countries is unlikely to be different. Even if householders are educated about the problems that are likely to be caused if stormwater run-off is introduced into sewers, some will still connect their yard or roof water into the sewer. For example, in low-income areas in Brasília and Natal around a quarter of households discharge some stormwater into their simplified sewer

(Sarmentos, 2000). In other cases, people will take the path of least resistance when faced with the possibility of flooding. For instance, it is not uncommon for people in Pakistan to lift manhole covers to allow water to run away into the sewers during and after storms.

So, it would appear to be unrealistic to design simplified sewerage systems to be completely separate. Where surface water drainage is a major problem, greater attention to the alternatives will have to be paid at the design stage; for more detailed information on planning for stormwater drainage, reference should be made to Kolsky (1998).

3.4.3.7 Minimum cover:

Cover is required over a sewer for three reasons:

(1) To provide protection against imposed loads, particularly vehicle loads,

(2) To allow an adequate fall on house connections, and

(3) To reduce the possibility of cross-contamination of water mains by making sure that, wherever possible, sewers are located below water mains.

Simplified sewerage should be designed with the objective of minimizing cover by locating sewers away from heavy traffic loads and as close as possible to existing sanitary facilities. In most cases, the loading criterion will be more critical than that to ensure adequate falls on house connections. The minimum cover criteria adopted will depend on local factors, in particular on the pipe material used (Sinnatamby, 1986).

3.4.4 Design over-riding parameters

3.4.4.1 Minimum sewer diameter:

It is necessary to specify a minimum sewer diameter because sewers transport wastewater which contains gross solids. As indicated in chapter 2, there is no theoretical reason why the minimum sewer diameter should not be 100mm. However, statutory authorities tend to be conservative on this point: for example, the minimum acceptable sewer diameter in Cairo, Egypt, is 180 mm; while that in Pakistan is 230 mm. Engineers are often reluctant to change. Every effort should be made to introduce appropriate standards, but it may be necessary to accept a higher minimum diameter than is absolutely necessary. In such circumstances, it is best to seek what is possible rather than the ideal. For instance, the acceptance of a 150 mm minimum diameter would be a big step forward in Pakistan.

3.4.4.2 Minimum flow:

Conventional sewer calculations assume steady-state conditions. In practice, the flow in sewers at the upper end of the system is highly transient. The amount of flow at any time depends on the number of taps running to waste and WCs being flushed. By far the largest flows occur when a WC is flushed. A wave passes down the house connection and into the sewer, becoming attenuated all the time by the effects of friction. Of course, the attenuation will tend to be greater if there is any interruption to its smooth flow – for instance, where a house connection enters a connection chamber above the sewer invert so that flows from the connection have to drop into the main sewer.

3.4.5 Design output parameters-minimum sewer gradient

There is still considerable uncertainty about the factors that influence solids deposition and movement in sewers. Research suggests sewers laid at flat gradients can remain free of settled solids even at very flat gradients. An example is provided by Gidley (1987), who reports on 6 and 8 inch (150 and 200 mm) diameter sewers laid at gradients of 0.11 and 0.2 percent (i.e. 1 in 900 and 1 in 500) in Ericson, Nebraska. The scheme served 80 households, a school and several commercial establishments; no operational problems occurred during 1976-1987, and there was no special maintenance. Lillywhite and Webster (1979) investigated the operation of a hospital drainage system in the United Kingdom, much of which had been laid to very flat gradients. They found that blockages rarely occurred except at points where there were faults in construction (for example, badly aligned sewer pipes) that broke the smooth flow in the sewer. Their conclusion was that poor construction quality is likely to have a bigger effect on the performance of a sewer than its gradient.

Both these systems can be assumed to have been essentially separate with no possibility of the entry of stormwater. Ackers *et al.* (1996) found that steeper gradients were necessary to avoid siltation in combined sewers receiving occasional high-sediment loads associated with stormwater flows. What do these findings suggest for the design of simplified sewerage systems? The first point is that the minimum permissible sewer gradient should be related to the construction quality – the better the quality, the flatter the allowable gradient. The second is that flatter slopes will be possible if stormwater, and the silt loading associated with it, can be excluded from sewers or trapped in a gully before entering the sewer.

Methods for calculating the minimum sewer gradient were discussed in previous chapter. The key parameter in determining the theoretical minimum gradient is the value adopted for minimum tractive tension. If the sewer can be constructed to a high standard and most stormwater can be excluded from the sewer, a value of 1 Pa can be used. This will give a minimum self-cleansing gradient of 1 in 213.

In situations where in practice it is considered that a minimum gradient of 1 in 200 is difficult to achieve, especially in flat areas if pumping is to be avoided, the designer is faced with two options:

(1) Accept that some siltation will occur and design the sewer on the assumption that it will have to be regularly desilted; or

(2) Provide interceptor tanks on all house connections to remove all but the smallest and lightest solids, i.e. design the system as a settled sewerage system (Otis and Mara, 1985; Mara, 1996). This allows much lower gradients to be used, but the system will eventually fail if the interceptor tanks are not desludged at the correct frequency.

3.4.6 Design of condominial sewers

This section details the steps necessary to prepare design information for a condominial sewer system to be input into the design program. It uses the example of a module forming part of a new sites-and-services housing scheme.

Figure 3.3 shows this module, together with a sewer layout to serve it. Plot boundaries are represented by thin lines and sewers by thick lines. No access points are shown at this stage. The plot sizes are small, representing typical practice in a new sites-and-services scheme. The five cul-de-sacs are relatively narrow lanes that are not intended for vehicular traffic. Sewers are proposed along the centres of these pedestrian lanes. Elsewhere inside the module, sewers are alongside the sides of streets, as close as possible to the front plot lines. The housing module fronts onto a main street, along which runs a public collector sewer. The larger plots that face onto the main street are connected to a local sewer that runs under the pavement, rather than directly to the collector sewer.

All the sewers serving the housing module thus form a condominial system that is self-contained and can be analyzed and designed regardless of the arrangements that are made elsewhere.

Similar arrangements, but including back-yard and/or front-yard sewers, could be adopted for a scheme with considerably larger plot sizes.

This is, of course, a very regular layout. In practice, many layouts will be less regular with some interconnections between different housing areas so that the limits of each 'condominium' may be more difficult to define. Nevertheless, the basic approach described here is valid for these more complex situations.

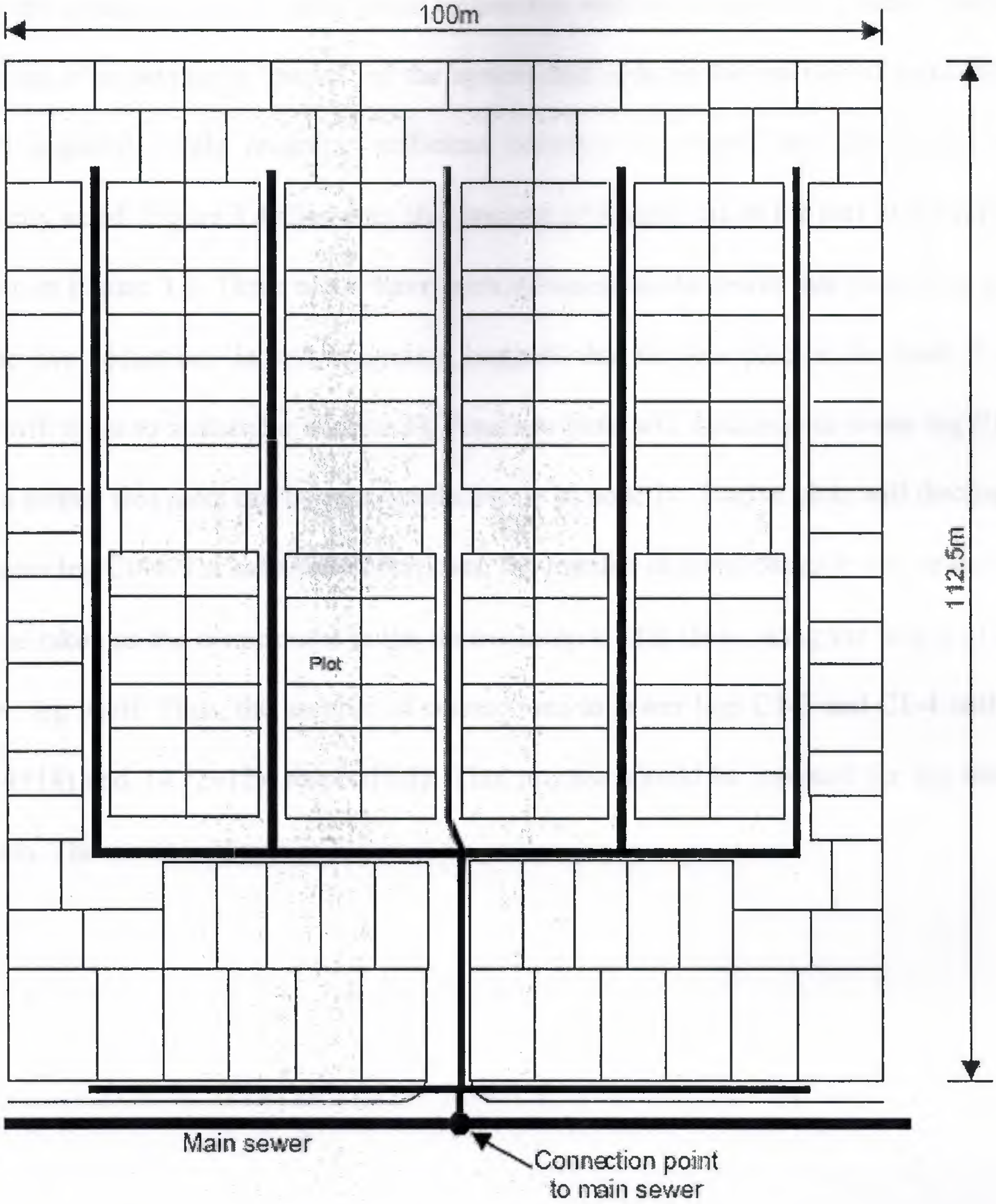

Figure 3.3 Sewer layout for a typical sites-and-services housing module.

The first step in the design process is to represent the system as a series of sewer 'legs' running between junctions or 'nodes'. In theory every house connection could be a node, but this would require a large number of calculations. The actual calculations are not a problem for the PC-based design program, but data entry would take a considerable amount of time. Fortunately, such a detailed approach is not necessary since the change in flow at each house connection will be infinitesimally small. Rather, the need is to develop a 'model' of the system that reduces the amount of calculation effort required, while retaining sufficient accuracy to ensure that the sewers are correctly sized. Figure 3.4 illustrates this process of simplification for part of the layout shown in Figure 3.3. Three nodes have been assumed on the sewer that runs along one of the five pedestrian 'lanes'. Inspection suggests that the four plots at the head of the lane will drain to a chamber at node J3. Fourteen plots will discharge to sewer leg C1-3 and a further two plots can be connected directly at node J4. Twelve plots will discharge to sewer leg C1-4. For calculation purposes, the number of connections to any sewer leg can be taken as the connections at the upstream node plus those along the length of the sewer leg itself. Thus, the number of connections to sewer legs C1-3 and C1-4 will be 18 (4+14) and 14 (2+12), respectively. This process should be repeated for the whole system. The result is shown in Figure 3.5.

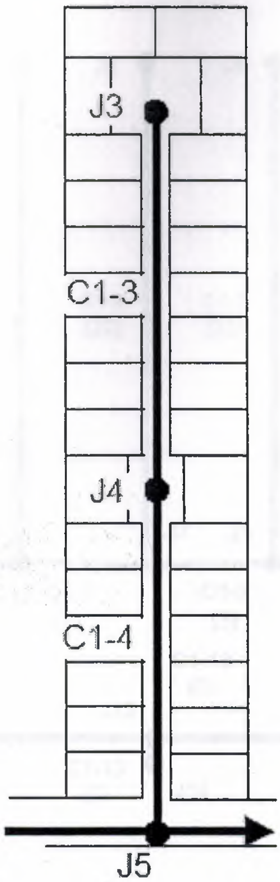

Figure 3.4 Sewer divided into legs running between nodes.

The PC-based design program will work whatever the numbering system, but interpretation of the results will be easier if there is some logic to the numbering system. With this in mind, the nodes and sewer legs have been numbered starting from the head of the left hand sewer.

The numbering system used for the sewers indicates that a condominial system, rather than public collector sewers, is being designed.

The figures given in brackets beneath the sewer leg numbers in Figure 3.5 are the number of house connections along those legs of the sewer.

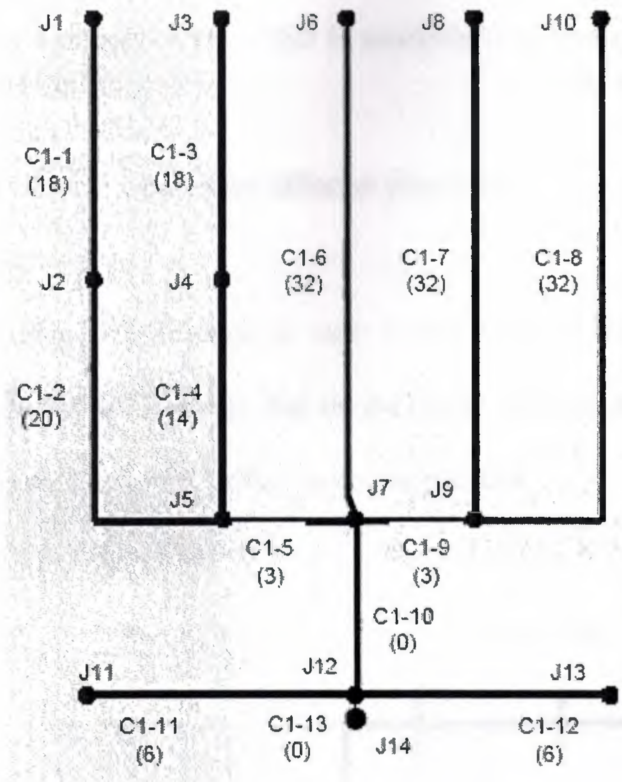

Figure 3.5 Numbering systems for sewer legs and nodes.

Note that the two lane sewers on the left of Figure 3.5 have intermediate nodes, which are omitted from the other three nodes. This has been done in order to test the sensitivity of the model to the number of nodes assumed. In practice, the intermediate nodes are not really required if the average ground slope along the sewers is fairly constant. Additional nodes should be inserted where there is a significant change in ground gradient since the sewer slope will have to be changed at this point and this need to be reflected in the calculations.

At this point there is much of the information required to input the sewer system into the PC-based design program. Additional information on the sewers themselves is required as follows:

(1) The lengths of all sewers – obtained by scaling off from the layout drawing.

(2) The ground level at each node – this is available from the physical survey of the area.

(3) The minimum allowable cover for different situations.

The normal procedure will then be to start at the head of the system, in the case illustrated in Figure 3.5 at J1 or J10, and set the sewer invert at that point such that the cover is the minimum allowable for the particular situation.

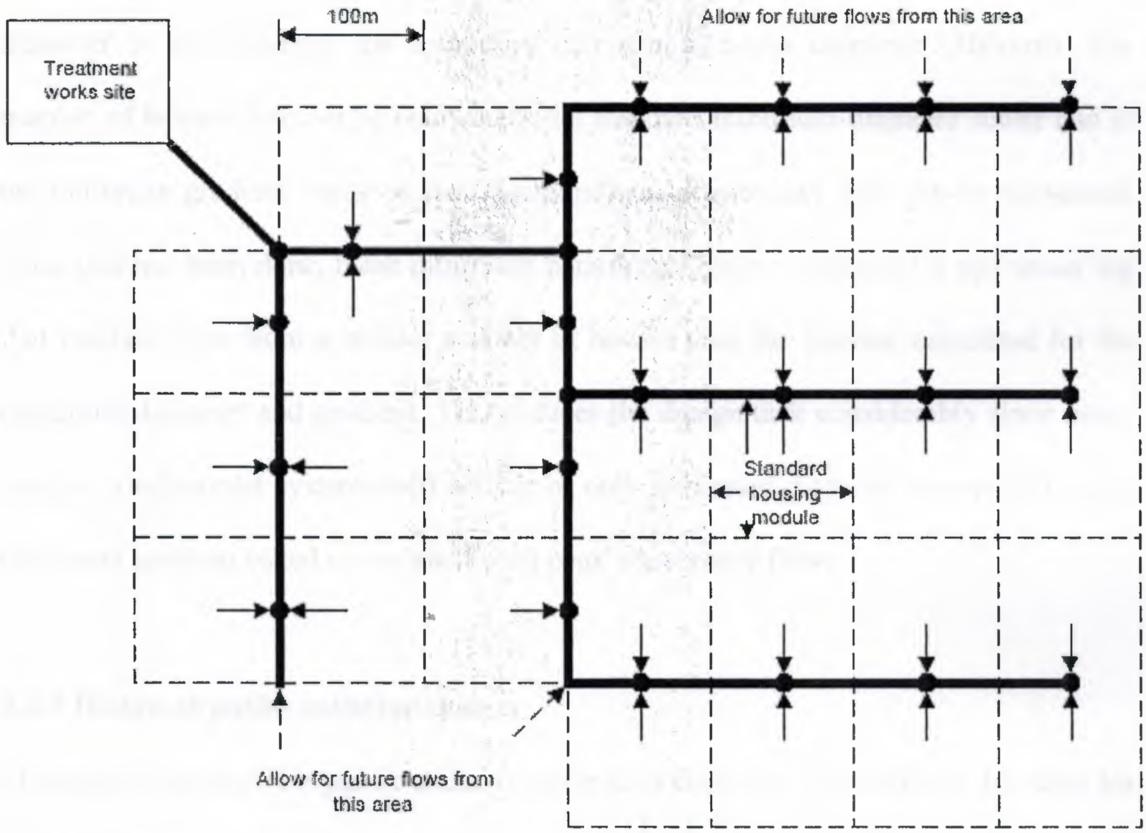

Figure 3.6 layouts for public collector sewers for a sites-and-services housing scheme.

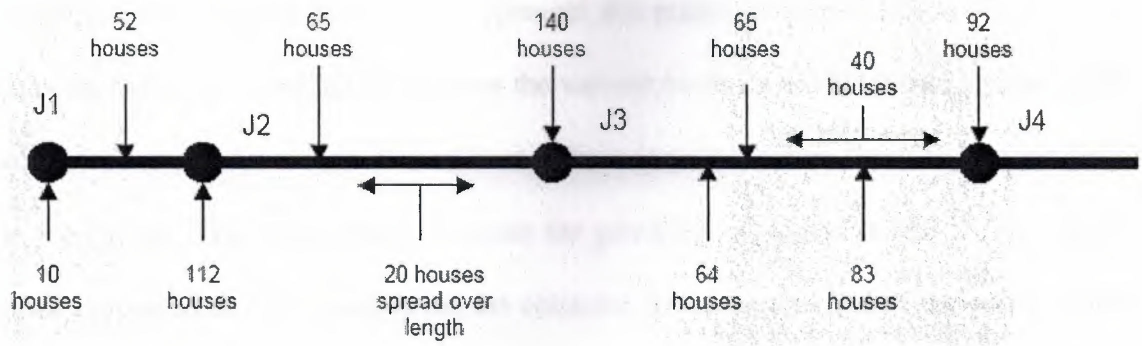

Figure 3.7 selection of node location.

As the design proceeds, it will be found that the slope of many sewers near the head of the system will be governed by the minimum wastewater flow (1.5 l/s), while their diameter is governed by the minimum permissible sewer diameter (100mm). The number of houses that can be connected to a standard minimum-diameter sewer laid at the minimum gradient based on the minimum peak wastewater flow can be calculated. Once this has been done, these minimum parameters can be assumed for any sewer leg that receives flow from a smaller number of houses than the number calculated for the minimum diameter and gradient. This reduces the design task considerably since many smaller condominial systems will consist of only minimum-diameter sewers laid at the minimum gradient based on the minimum peak wastewater flow.

3.4.7 Design of public collector system

The design approach for public collector systems is essentially the same as that used for condominial systems in that, for calculation purposes, the sewer system is divided into legs connected at nodes. Figure 3.6 illustrates a sewer layout for a sites-and-services scheme based on the module that has already been used to illustrate the design of a condominial system. The dashed lines indicate the borders of individual housing modules and the thick black lines represent the public collector sewers. The arrows indicate the points at which flows from the various modules are discharged to the public collector sewers. Arrows on dashed lines indicate possible future flows to be considered in the design. The black circles indicate the positions of nodes. It will be seen that a node is located at each junction on the collector sewer system and at the points where flows from the modules discharge to the collector sewers. Any direct inflows to the collector sewer between nodes are assumed to be concentrated at the downstream node, as in the case of condominial systems.

This is a regular layout with inflows to the public collector sewers concentrated at nodes. In practice, most systems are more complex and it may be that inflows are spread along the length of the collector sewer rather than concentrated at one point, as shown in Figure 3.7. In such situations, it is necessary to use judgement in the selection of node locations. Figure 3.7 suggests that:

· nodes should be located at all points where there are relatively large inflows to the sewer; and

· closer node spacing is needed near the head of the system.

4. PC-BASED SIMPLIFIED SEWER DESIGN

4.1 INTRODUCTION

In this thesis, simplified sewer design is used to generate possible outcomes of the method by means of a computer program. The program is written in the university of Leeds, Department of civil engineering by prof. Duncan Mara.

The program aims to speed up the design calculation, provide different design configurations at a short time interval, and helps those people whose purpose is training or learning of simplified sewer design.

In order to run the program, the main requirement is the description of sewer network. A sewer network consists of pipes which are linked to each other. This linkage can only be constructed in a tree type manner. The design, therefore, should always start from the most downstream point branching at junctions to several upstream ends. The constructed network may be split into subnetworks, termed as "sub-net". These "sub-net" may later join each other at joints called "drop". In summary a network consists of one or more sub-nets which may join at drop junctions.

The basic layout of the design requires three main informations related to the environmental conditions. The length of the sewer to be designed, the number of people connected to it and the ground levels of downstream and upstream points are also necessary.

4.2 DATA ENTRY/EDIT PROCEDURE

Figure 4.1 presents a series of tables and boxes which are used to describe the sewer network layout, and usage requirements.

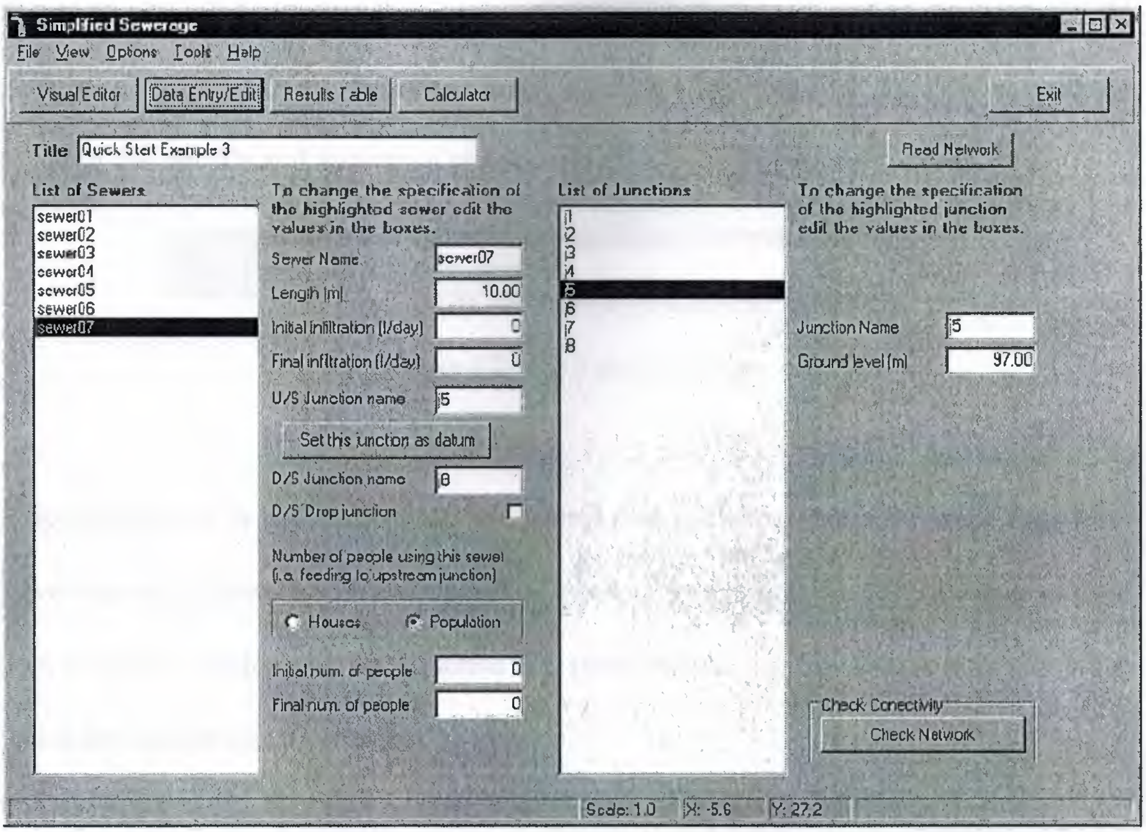

Figure 4.1 The Data Entry/Edit screen with example data.

In the above figure, the necessary lists and boxes are filled to represent the possible numbers that can be used as an entry data. These data entry files all have the extension ".snt". The."snt" file contains the network description as well as the calculation details – sizes, slopes, depths etc. – if they have been calculated. These files may be easily read, saved and read back into the program. From this data entry screen the button labeled "Read Network" will open a dialog allowing to choose one of these files and load it into the program.

There are two alternatives; either to use an old network file or create a new one. A new network or project title can be edited in the Title Edit box (Figure 4.2). This title will be saved in the .snt file and appear on graphs and other printouts of the solution.

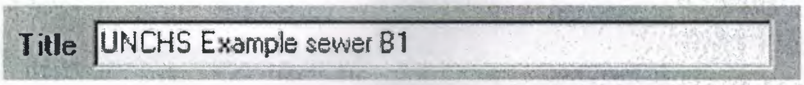

Figure 4.2 The Title Edit box.

A sewer network is made up of named sewers that join at named junctions. Thus each sewer has an upstream and a downstream junction. Sewers and junctions each have their own properties which in total describe the sewer layout and the wastewater flows for which the design will be made.

This screen provides an alternative to the Visual Edit screen for entering, displaying and editing all of the network and demand properties. The names of all the sewers are listed in "List of sewers" (Figure 4.3). Clicking on any one of these displays, the properties of the clicked sewer, in the edit boxes, at the right part of the menu.

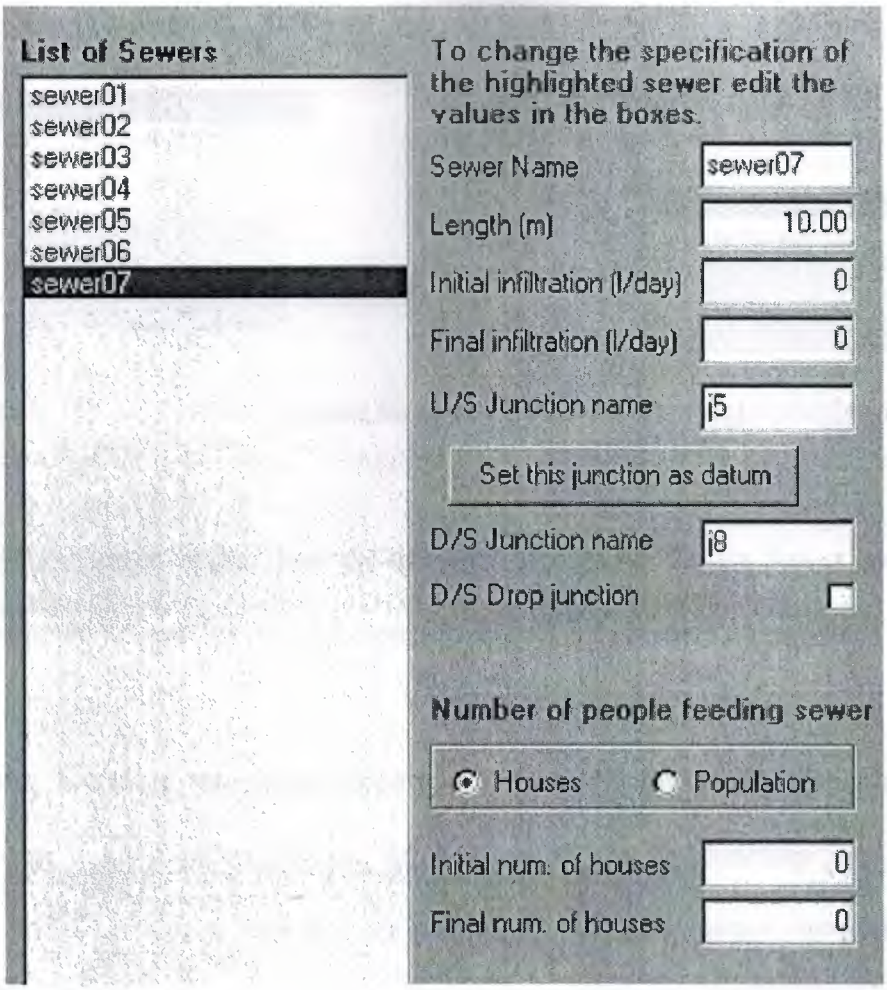

Figure 4.3 The Sewer List Box and Sewer Data Edit boxes.

A new sewer can be added to the network by pressing the right mouse button in the sewer list box and selecting Add Sewer (Figure 4.4). The sewer is given a unique name and default property values which must be edited to fit in the network.

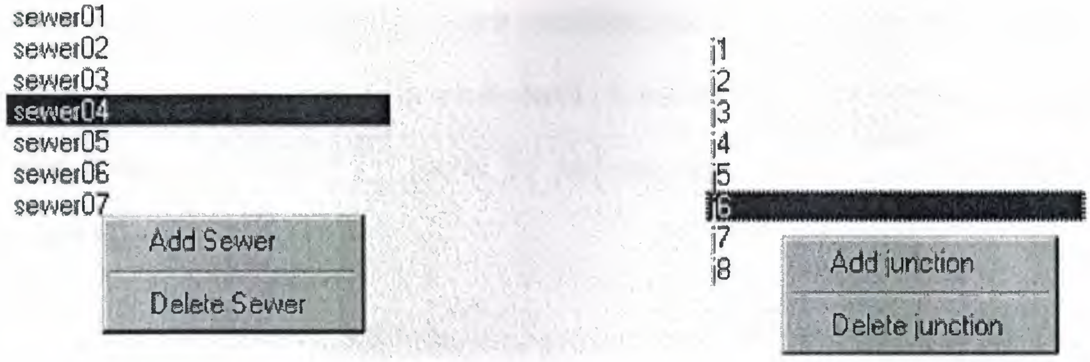

Figure 4.4 The Add/Delete options for Sewer/Junction when right-clicking the list.

To remove a selected sewer from the network chooses the Delete Sewer option. You will be prompted to confirm this deletion to help prevent accidental removal.

Like sewers, junctions also have properties that are displayed when clicking on the junction name in the junction list box. Again, as for sewers, the junction properties may be edited and the changes recorded by pressing the Apply Change button below the Junction Data Edit box.

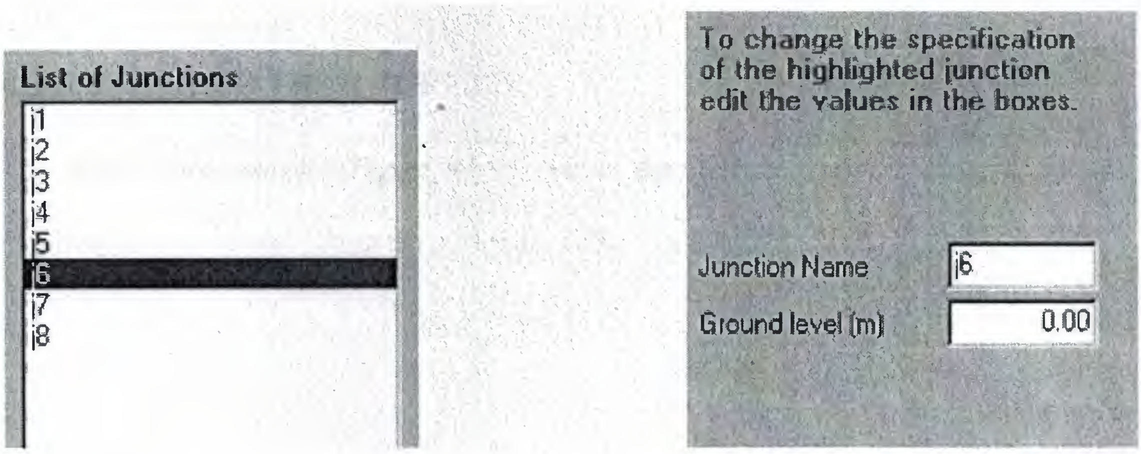

Figure 4.5 The Junction List and Junction Data Edit boxes.

Once the sewers and junctions have been specified, the sewer network description is complete. It is necessary to perform some checks to ensure that it is described correctly and that it does not contain any loops. To perform these checks, press the "Check Network" button.

A network may be valid and correct with junctions that are not connected anywhere. It may be that these are placed with the expectation that new sewers will join them later. On the other hand it may be better to remove them to keep the network tidy. To remove all unconnected junctions choose the Options menu so that the item "Delete Unused Junctions" is checked before pressing the "Check Network" button.

Once the network has been entered and checked, it should be saved before going any further. As mentioned above, sewer networks should be saved to. snt files. After entering and saving the network, go on to the Results Table screen by pressing the Results Table button on the top tool-button bar. A solution will be automatically calculated and displayed.

4.3 RESULTS TABLE SCREEN

The results table screen (Figure 4.6) presents the designed sewer network. All the required data for constructing the sewer are shown in this table.

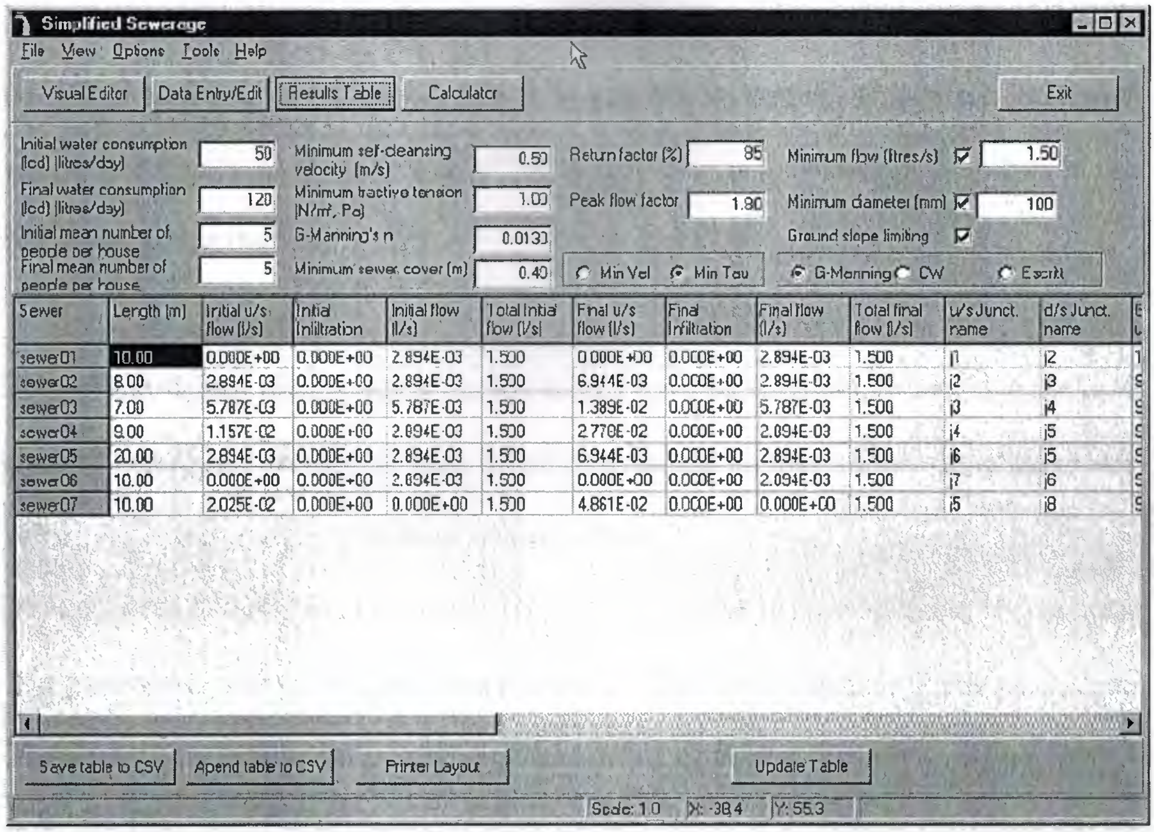

Figure 4.6 The Results Table screen.

If a sewer network has been previously entered – by reading from an existing file, via either the Data Entry/Edit or Visual Editor Screens – then the solution will be automatically shown when this screen is displayed. This screen can not edit the values. To change the results to a different solution one must edit the initial data and/or the calculation parameters.

Each column of the table has a heading, which identifies the parameter found for each sewer. The data for each sewer are written on a single row, and the sewer is identified by its name in the first column headed "Sewer".

As well as displaying the solution, this screen also allows the setting of parameters for the whole network (the Data Entry/Edit screen allows setting of data for individual sewers of the network). These "global" settings are made by changing the values in the edit boxes above the table.

The default values which appear on this screen have been chosen to be those (or very close to those) that should be used. Great care should be taken when using parameter values that differ greatly from these default values.

The parameter setting will now be explained. The first column of four edit boxes (Figure 4.7) deals with water consumption which, together with the population setting from the Data Entry/Edit screen of each sewer (Figure 4.3) defines the water use. If "Population" was chosen to define use, then only the top two boxes – initial and final consumption – need to be set. However, if "Houses" was chosen, then the lower two boxes – the initial and final mean number of people per house – are also required to define the total number of people using the sewer and hence the total water use.

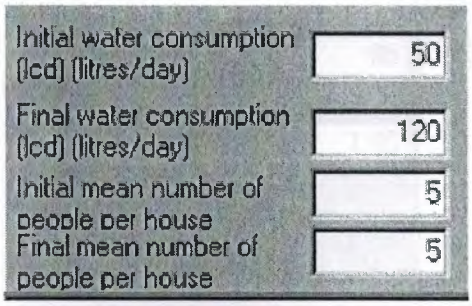

Figure 4.7 Water consumption data edit boxes.

The rest of the settings deal with how the sewer design calculations are to be performed.

Figure 4.8 shows the options to set either the minimum self-cleaning velocity or minimum tractive tension or both (Only one solution will be displayed based on one of these methods; however both calculations are performed, so both values should be entered.). The value for Gaukler-Manning's *n* (shown as G-Manning's *n*) may also be set in the next edit box. The minimum cover of sewer is the depth set for the upstream junction that has been designated as the datum junction. All other depths will be calculated relative to this according to the calculated sewer gradients.

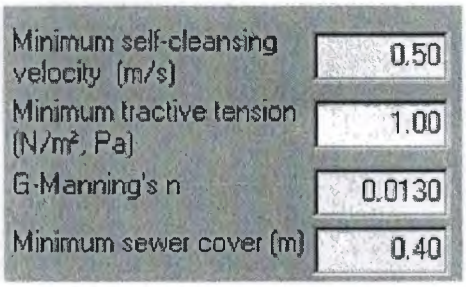

Figure 4.8 Calculation parameters.

Figure 4.9 shows the edit boxes for Return Factor (entered as a percentage) and Peak Flow Factor. Also shown is the choice of calculation for which results will be presented – "Min Vel" refers to the calculation based on minimum self-cleansing velocity, while "Min Tau" refers to that based on minimum tractive tension.

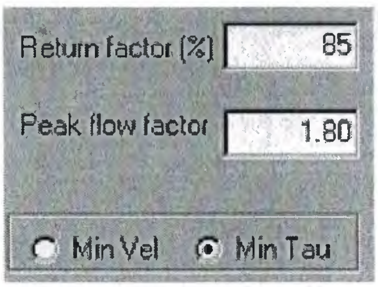

Figure 4.9 Calculation parameters.

It may be desirable to set some design limits for the calculation. Three check boxes are provided for this (Figure 4.10). Ticking the Minimum Flow check box means that if the sewer demand is less than the minimum entered in the edit box to the right, then this minimum is used for the sewer design. Ticking the Minimum Diameter check box means that no sewer of diameter less than the entered minimum will be used. Ticking the Ground Slope Limiting box means that the minimum slope of the sewer will not be less than the ground slope.

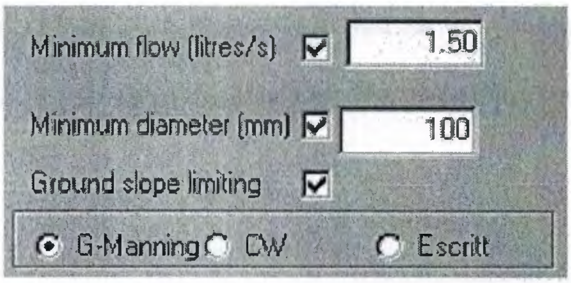

Figure 4.10 Limiting values and Velocity of Flow equations.

Also shown in Figure 4.31 is the choice of velocity of flow equation – Gaukler-Manning (shown as G-Manning), Colebrook White (shown as CW) or Escritt.

When changes have been made to any parameters, to see what effect these changes have on the solution the Update Table button must be pressed. The table will then be updated with the new results.

4.4 CALCULATOR SCREEN

This screen (Figure 4.11) presents the detail of the calculation for an individual sewer. Opening the screen automatically displays the design parameters for the current sewer of the Results Table (the last sewer row to be clicked on or highlighted on the Visual Edit screen).

This screen allows the calculation to be performed with new demand data or new calculation parameters, so that changes in design can be investigated. Any changes made to this screen are not transferred to the Results Table of sewer data.

To the left hand side of the screen are displayed all the design parameters; these have been taken from those previously entered on the either the Data Entry/Edit or Results Table screens.

The first eight of the edit boxes for the design parameters deal with the sewer's initial and final demand in terms of population and water consumption (Figure 4.12).

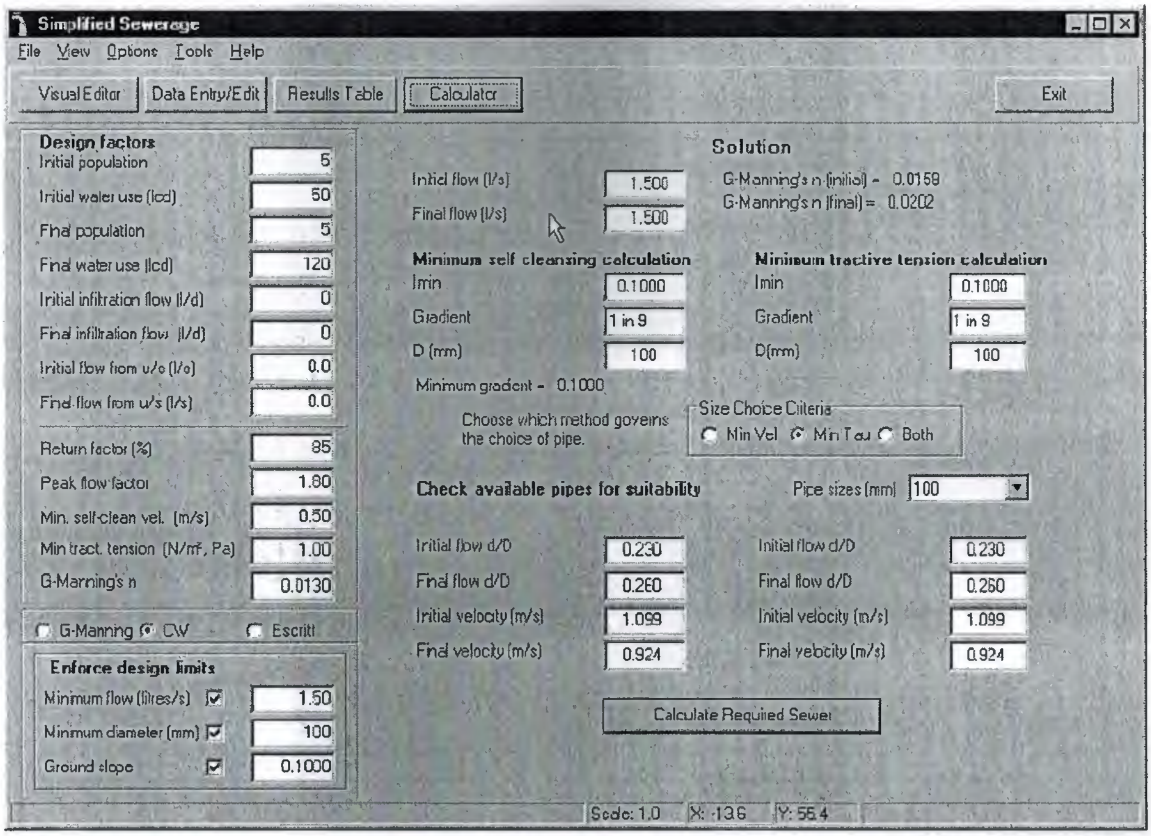

Figure 4.11 Sewer Calculation screen.

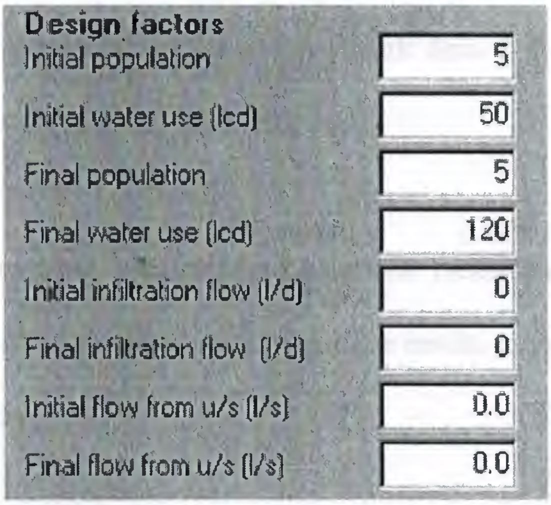

Figure 4.12 Water consumption settings.

Below these the design parameters (Figure 4.13) and design limits (Figure 4.14) are displayed.

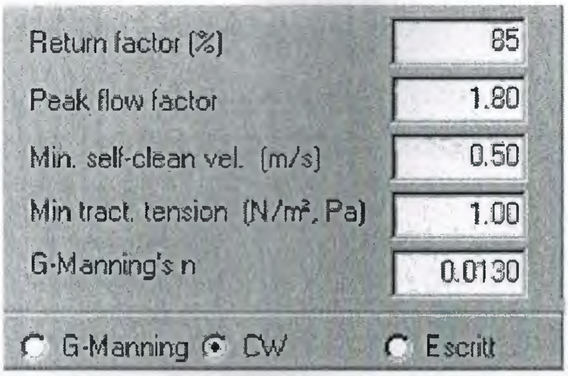

Figure 4.13 Design parameters.

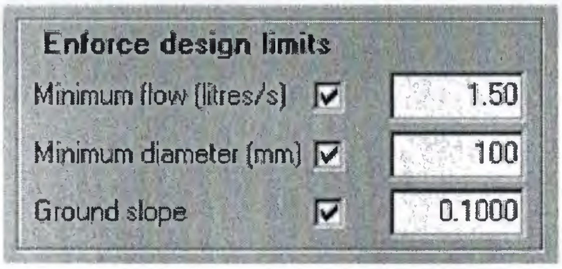

Figure 4.14 Design limits.

Note that the ground slope can be changed here; this is not possible on the Results Table form as that would entail changing junction/network data. It is possible here as this screen is only concerned with a single sewer.

The Calculator screen shows results of calculations based on both minimum self-cleaning velocity and minimum tractive tension. The results are displayed side-by side, as shown in Figure 4.15.

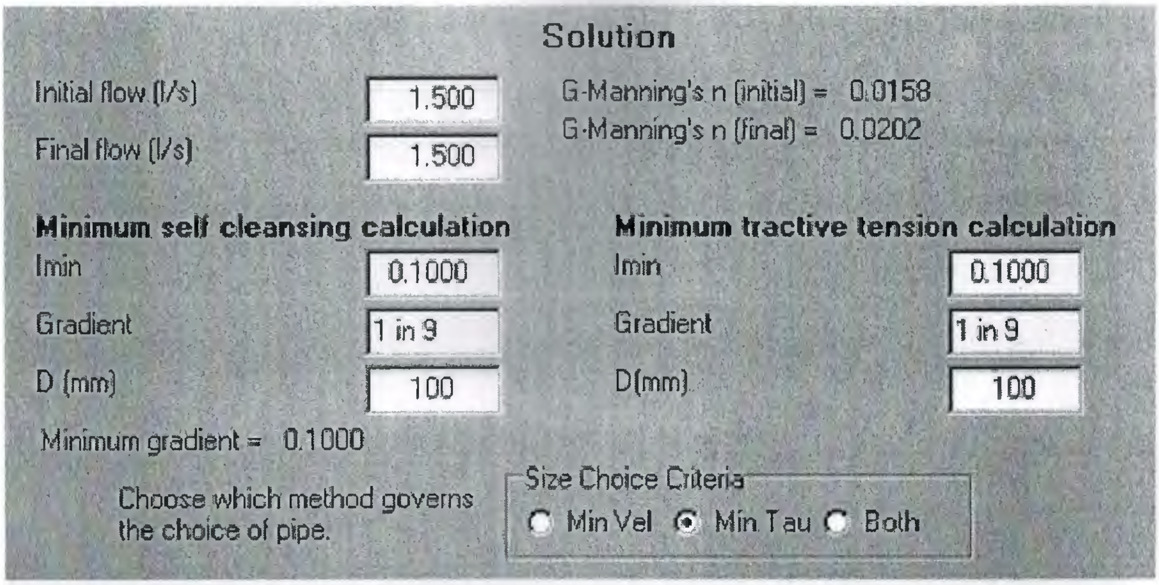

Figure 4.15 The tow solutions side by side

The calculated gradient and diameter of the sewer for both calculations are shown in Figure 4.15 (the diameter may be the set to minimum if that option was chosen). If the gradient is limited by the ground slope, then this limit will be shown as the minimum gradient.

The total initial and final flows are calculated and displayed at the top of the screen. If either the Colebrook-White or Escritt equations have been chosen, then the *equivalent* Gauckler-Manning's n is displayed next to these (Figure 4.16).

In Figure 4.17 the chosen sewer (from the list of available sewers) is displayed and, to enable a check on the solution, so are the d/D and velocity values for the initial and final flows.

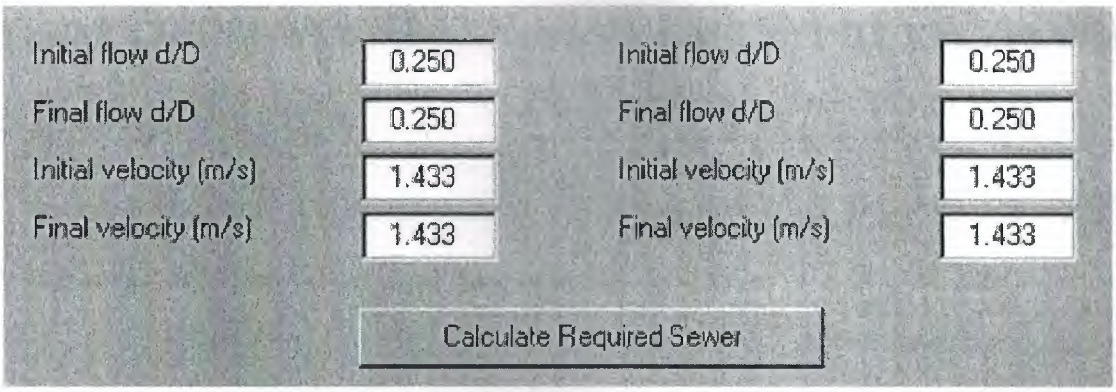

Figure 4.16 Check sewer design.

Figure 4.17 Chosen sewer selector.

5. SIMPLIFY SEWER APPLICATION; CASE STUDY IN LEFKOSA

5.1 INTRODUCTION

The case study about the simplify sewer method is applied to the Taskinkoy region, which is one of the most populated area of capital lefkosa.

Initially, the number of houses, the location of houses and streets are figured out by the help of maps, in 1/1500 scale. Those maps are digililized by the help of software GIS, in order to obtain surface elevation of the site. It's preferred to use the number of houses during the analyses, since the population at that region is not uniform. This non uniformity is due to the variable student residence at the region.

In Figure 5.1 for design I and Figure 5.2 for design II the detailed surface elevations which are given in 0.5m intervals are plotted. Also the location of houses can be seen in Figure 5.1 and Figure 5.2. The number of houses available at the region is 180, and the total distance under the consideration is (1360) m.

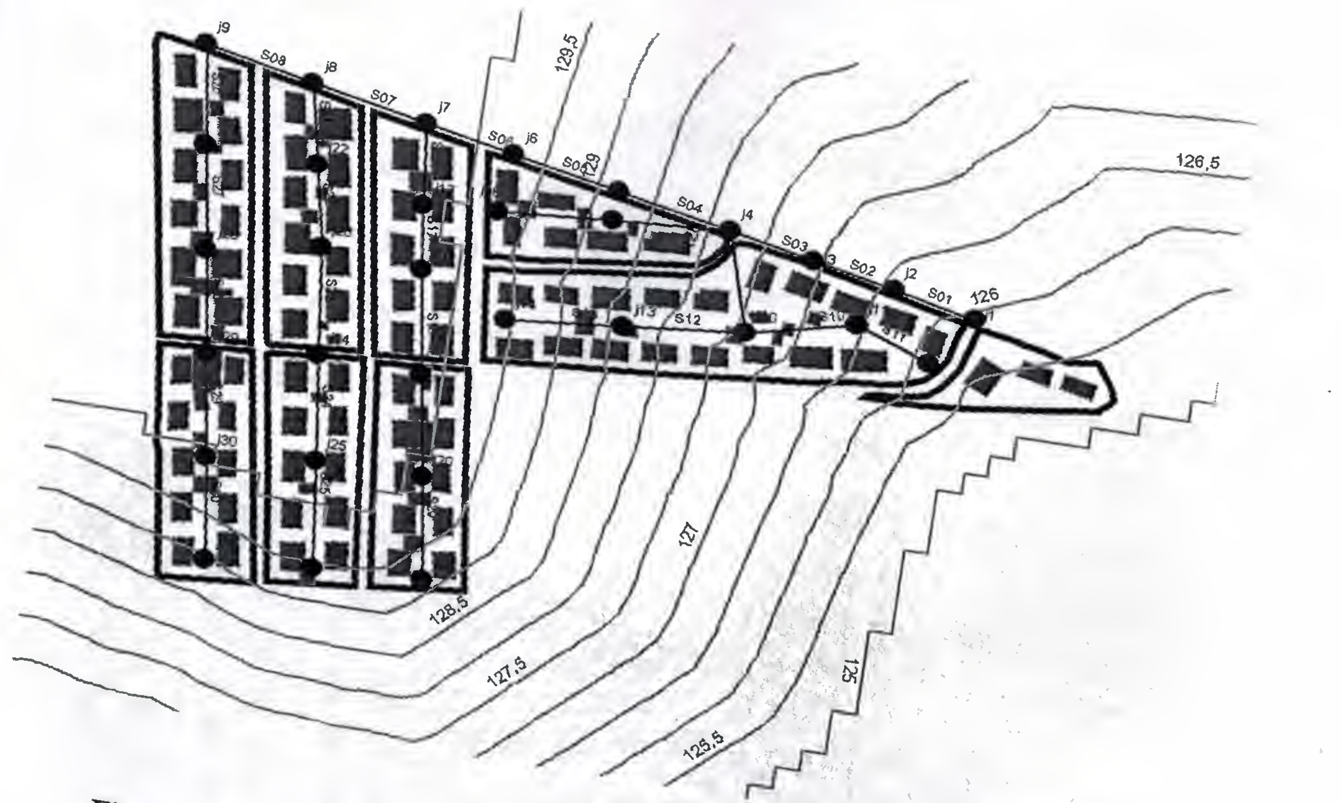

Figure 5.1 surface elevations which are given in 0.5m intervals, design I.

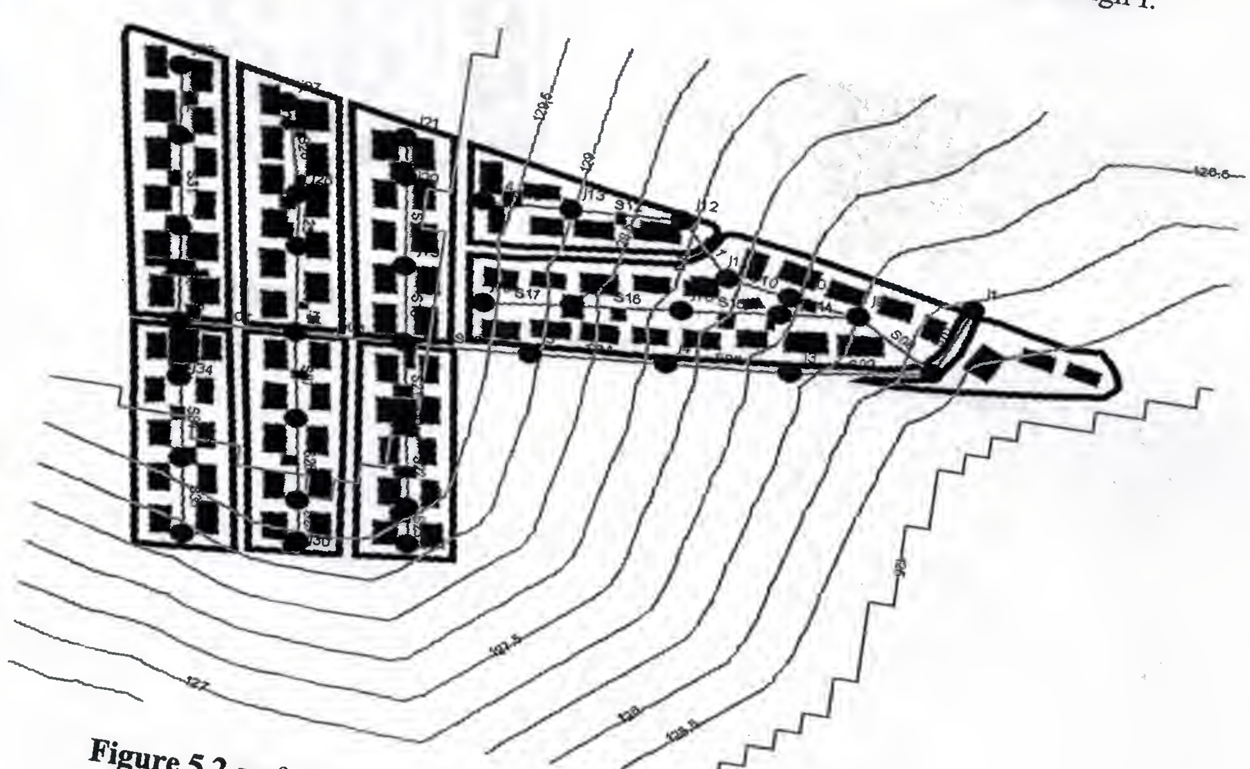

Figure 5.2 surface elevations which are given in 0.5m intervals, design II.

5.2 DATA INPUT

All the necessary information taken from the GIS software is an input value for the program "PC-based simplified sewer design program" so that the analyses can be carried out efficiently. First of all the length of sewer; number of houses that will be connected to the sewer and the ground levels of the junctions point are entered. There application can be seen in Figure 5.3 and Figure 5.4.

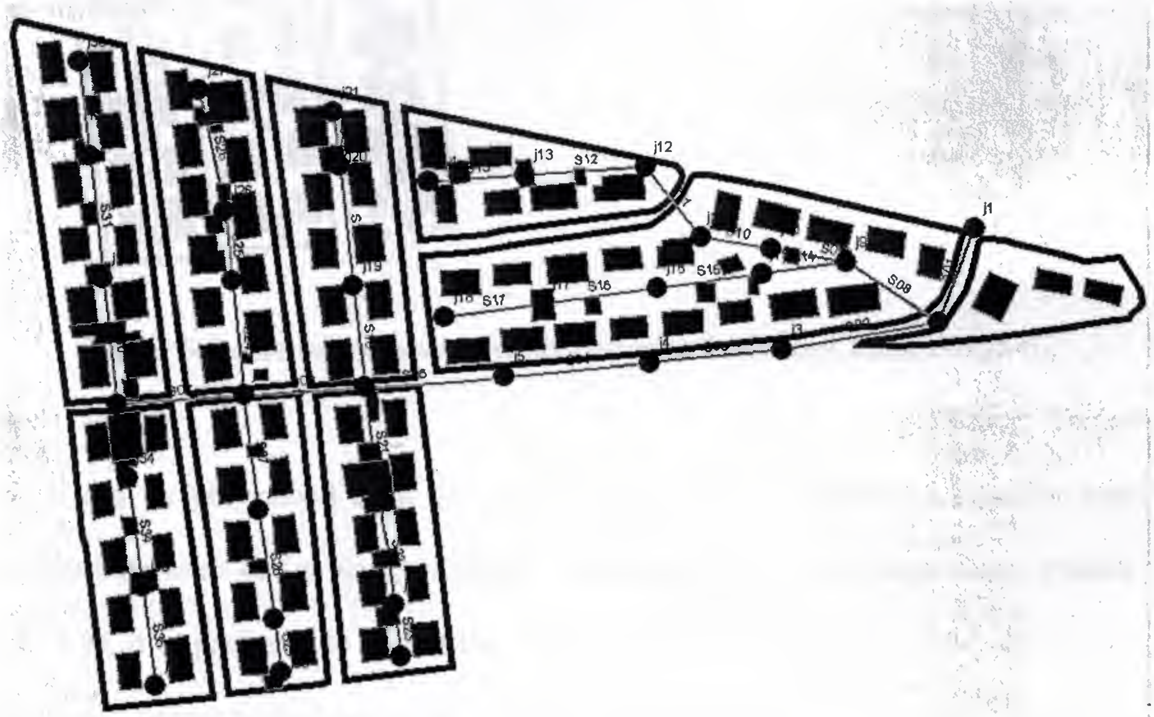

Figure 5.3 sewer network designs at Taskinkoy, titled, design I.

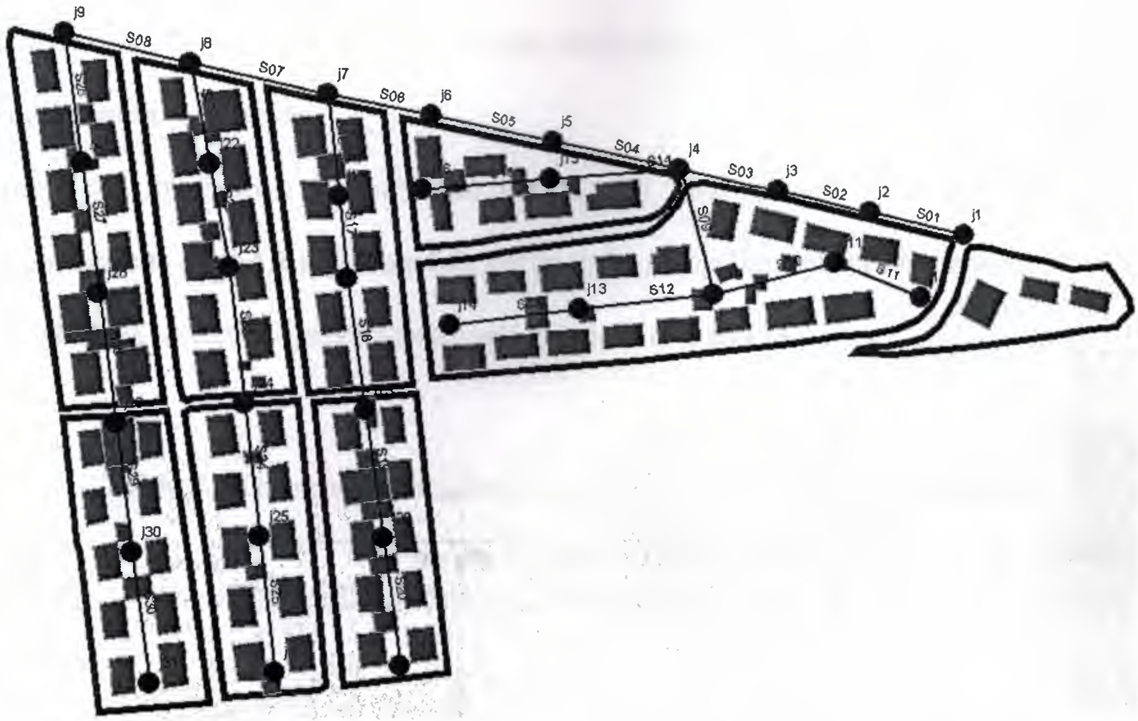

Figure 5.4 Alternative sewer network design at Taskinkoy, titled, design II.

As it can be understood from the figures, the method application is done for tow different network design named design I and design II. In one of there design studies (fig 5.3), it is preferred to collect the network system at the center of the region and make sub connections through the backyards of the houses. By this way, it's tried to shorten the length of the individual connections to the main sewer. The approximate distances between each junction is around 30-50 meters.

In Figure 5.4 the network system is arranged to discharge the out flows through the main road, which is at the Northern part of the region. However the disadvantage of this design system is that the backyard connection takes long distances.

The data input for design I is given in table 5.1. The first column indicates the sewer name in between tow corresponding junctions. The representative length of each sewer is given in second column. Also in this table one can read the number of houses connected to each sewer.

Table 5.2 summaries the junction names and their ground level elevations. In accordance to table 5.1 and table 5.2, the sewer01 is pipeline network in between J1 and J2 where as sewer02 is pipeline network in between J2 and J3.

Table 5.1 The sewer name, length and the number of houses connected.

sewer name	length (m)	number of houses connected
sewer 01	36,51	4
sewer 02	58,27	4
sewer 03	49,3	5
sewer 04	54,844	5
sewer 05	51,1	2
sewer 06	45,45	0
sewer 07	46,204	0
sewer 08	39,82	3
sewer 09	28,14	3
sewer 10	26,24	1
sewer 11	33,1	2
sewer 12	45,271	3
sewer 13	35,42	6
sewer 14	31,36	0
sewer 15	39,3	2
sewer 16	43,413	4
sewer 17	36,6	4
sewer 18	37,873	8
sewer 19	44,213	8
sewer 20	19,94	4
sewer 21	41,29	8
sewer 22	39,63	8
sewer 23	18,884	4
sewer 24	42,901	8
sewer 25	26,1	5
sewer 26	45,65	9
sewer 27	42,64	8
sewer 28	40,28	8
sewer 29	19,78	4
sewer 30	47	8
sewer 31	45,3	9
sewer 32	34,773	7
sewer 33	27,52	6
sewer 34	40,2	6
sewer 35	36,994	8

Table 5.2 The junction name and the ground level for each junction.

junction name	Ground level (m)
J1	125,9362
J2	125,7905
J3	126,6079
J4	127,7044
J5	129,1237
J6	130
J7	130
J8	130
J9	126,4625
J10	127,0746
J11	127,5126
J12	128,1846
J13	129
J14	129,8167
J15	127,0446
J16	127,8075
J17	128,6518
J18	129,6216
J19	130
J20	130
J21	130
J22	130
J23	129,5922
J24	129,1974
J25	130
J26	130
J27	130
J28	130
J29	130
J30	129,445
J31	130
J32	130
J33	130
J34	130
J35	129,8319
J36	128,7604

The data input for design II is given in table 5.3. The first column indicates the sewer name in between tow corresponding junctions. The representative length of each sewer is given in second column. Also in this table one can read the number of houses connected to each sewer.

Table 5.4 summaries the junction names and their ground level elevations. In accordance to table 5.3 and table 5.4, the sewer01 is pipeline network in between J1 and J2 where as sewer02 is pipeline network in between J2 and J3.

Table 5.3 The sewer name, length and the number of houses connected.

sewer name	length (m)	number of houses connected
sewer 01	36,96	2
sewer 02	35,33	4
sewer 03	36,41	0
sewer 04	48,43	0
sewer 05	46,78	1
sewer 06	39,25	0
sewer 07	50,99	0
sewer 08	47,41	0
sewer 09	47,77	2
sewer 10	46,51	5
sewer 11	34,7	5
sewer 12	49,81	10
sewer 13	49,3	10
sewer 14	48,04	3
sewer 15	48,51	8
sewer 16	38,1	6
sewer 17	30,8	6
sewer 18	49,27	8
sewer 19	47,79	10
sewer 20	48,19	10
sewer 21	38,18	7
sewer 22	39,32	8
sewer 23	50,43	7
sewer 24	49,84	10
sewer 25	50,27	10
sewer 26	48,27	9
sewer 27	48,81	9
sewer 28	48,74	6
sewer 29	48,19	10
sewer 30	48,74	10

Table 5.4 The junction name and the ground level for each junction.

junction name	Ground level (m)
J1	125,8713
J2	129,3791
J3	126,9606
J4	127,8066
J5	128,8566
J6	129,7027
J7	130
J8	130
J9	130
J10	127,4526
J11	126,4625
J12	125,8252
J13	128,4039
J14	129,6216
J15	128,7967
J16	129,8167
J17	130
J18	130
J19	130
J20	129,9892
J21	129,1974
J22	130
J23	130
J24	130
J25	130
J26	129,445
J27	130
J28	130
J29	130
J30	130
J31	128,7604

All the data represented in the above table are displayed in "PC-based simplified sewer design program" by the help of Entry/Edit screen. There are two alternative to input the data. The data input can be either by using keyword, writing into the data entry screen, or by using mouse and drawing the network in visual editor screen is also helpful since junction point are already given, for visual checks.

The first thing to do before entering the sewer network data on the Data Entry/Edit screen is to draw the system of sewers and label each junction and sewer in a similar way to that shown in Figure 5.3, and Figure 5.4 this will make the job of data entry much easier.

To input the sewer data by using keyboard for the first network, first create the sewers by pressing right mouse button and choosing the "Add Sewer" option (see Figure 5.6). Do this 35 times. This will create 35 new sewers with the names S0, S1, S2... and S35, all with the same default data. These can now be edited to represent the network. Click on S0 in the list of sewers and change the sewer name to S1, the length to 36.51m, the upstream junction to j2 and the downstream junction to j1. Click on the "Houses" option so that a black dot appears next to the label "Houses" and enter 4 in both the initial and final number of houses edit boxes. The data in the edit boxes should now look like those in Figure5.6. In a similar way enter the data for S2, S3, S4... and S35, so that the data are as shown in Figures5.6.

What is left is the entrance of the junction name and elevation. Now enter the junction data. There should be a list of 36 junctions in the junction list.

Click on "j2" to display the junction data. Leave the name but enter the ground level as 125.79 m. The data should then appear as shown in Figure 5.6. In the same way enter the data for junction's j1, j3, j4... and j36.

The second alternative which is to be in visual editor screen is done by directly drawing the system and at the same time, entering the information of elevation, length of the sewer and the junction. This screen is given in Figure5. 5.

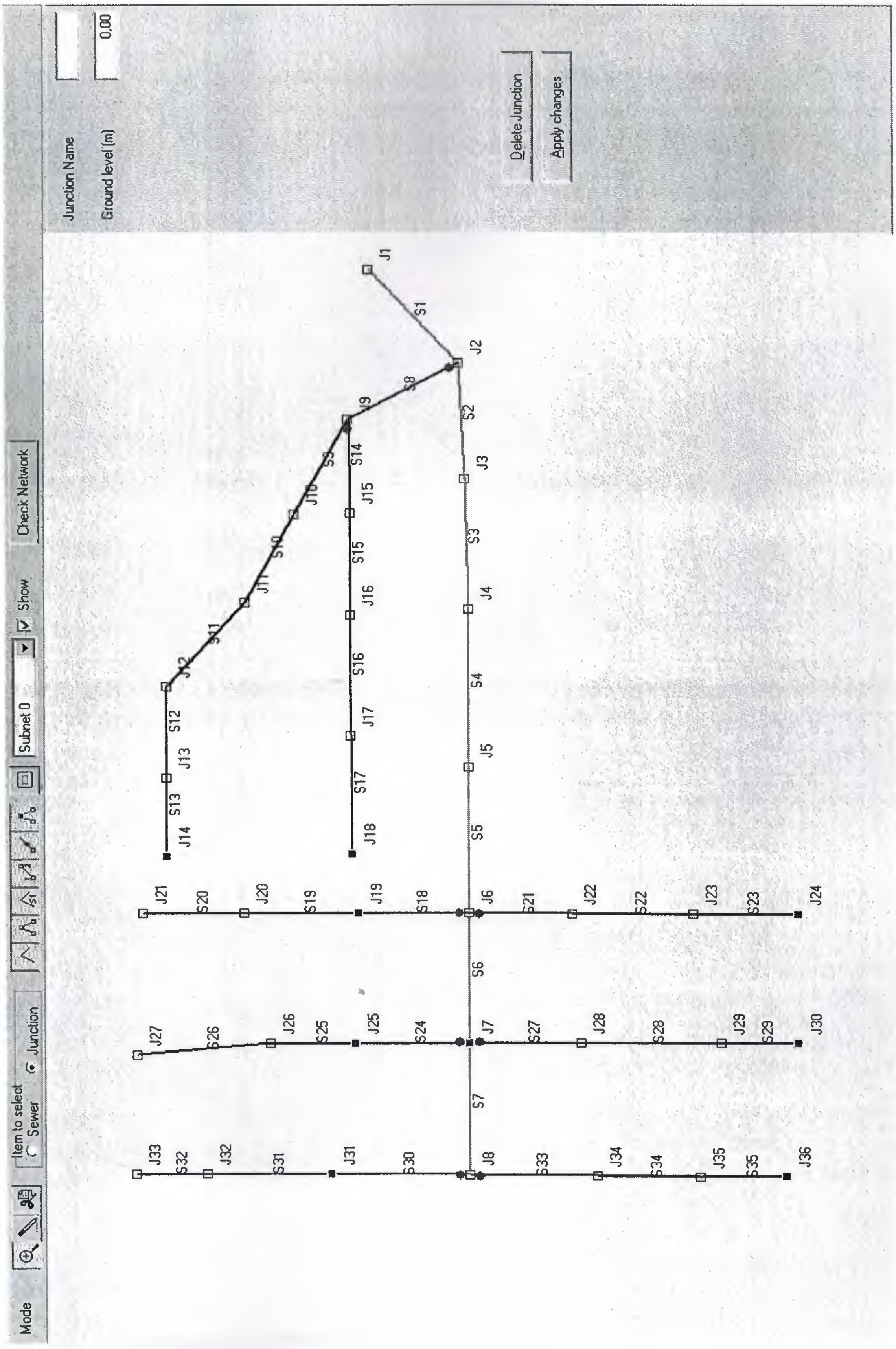

Figure5.5 The visual editor screen, design I.

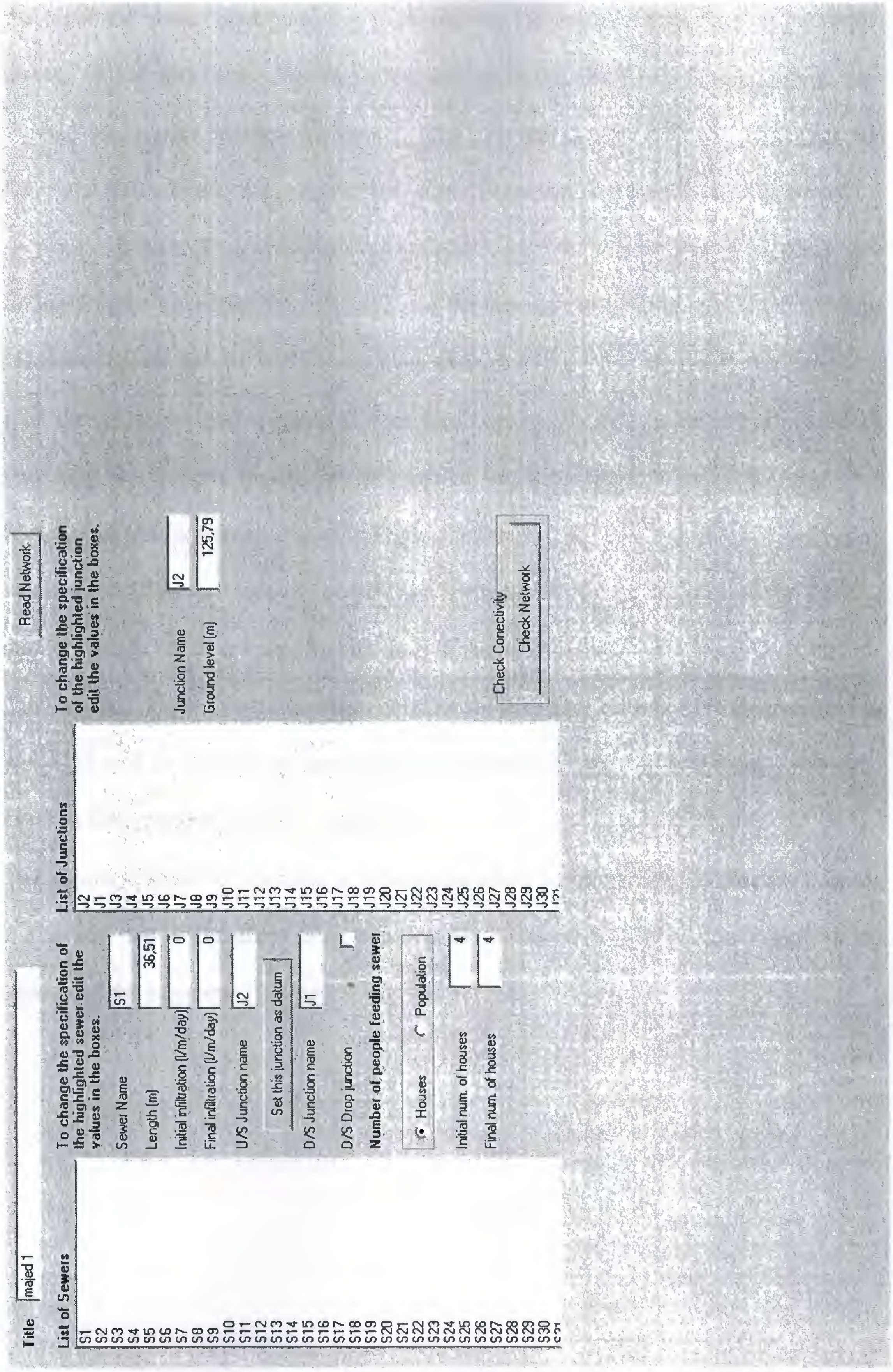

Figure 5.6 The data Entry/Edit screen, deaign I.

To input the sewer data by using keyboard for the second network, first create the sewers by pressing right mouse button and choosing the “Add Sewer” option (see Figure5.8). Do this 30 times. This will create 30 new sewers with the names S0, S1, S2... and S30, all with the same default data. These can now be edited to represent the network. Click on S0 in the list of sewers and change the sewer name to S1, the length to 36.96m, the upstream junction to j2 and the downstream junction to j1. Click on the “Houses” option so that a black dot appears next to the label “Houses” and enter 2 in both the initial and final number of houses edit boxes. The data in the edit boxes should now look like those in Figure 5.8. In a similar way enter the data for S2, S3, S4... and S35, so that the data are as shown in Figures 5.8.

What is left is the entrance of the junction name and elevation. Now enter the junction data. There should be a list of 31 junctions in the junction list.

Click on "j2” to display the junction data. Leave the name but enter the ground level as 125.8713 m. The data should then appear as shown in Figure 5.8. In the same way enter the data for junctions j1, j3, j4... and j31.

The second alternative which is to be in visual editor screen is done by directly drawing the system and at the same time, entering the information of elevation, length of the sewer and the junction. This screen is given in Figure 5.7.

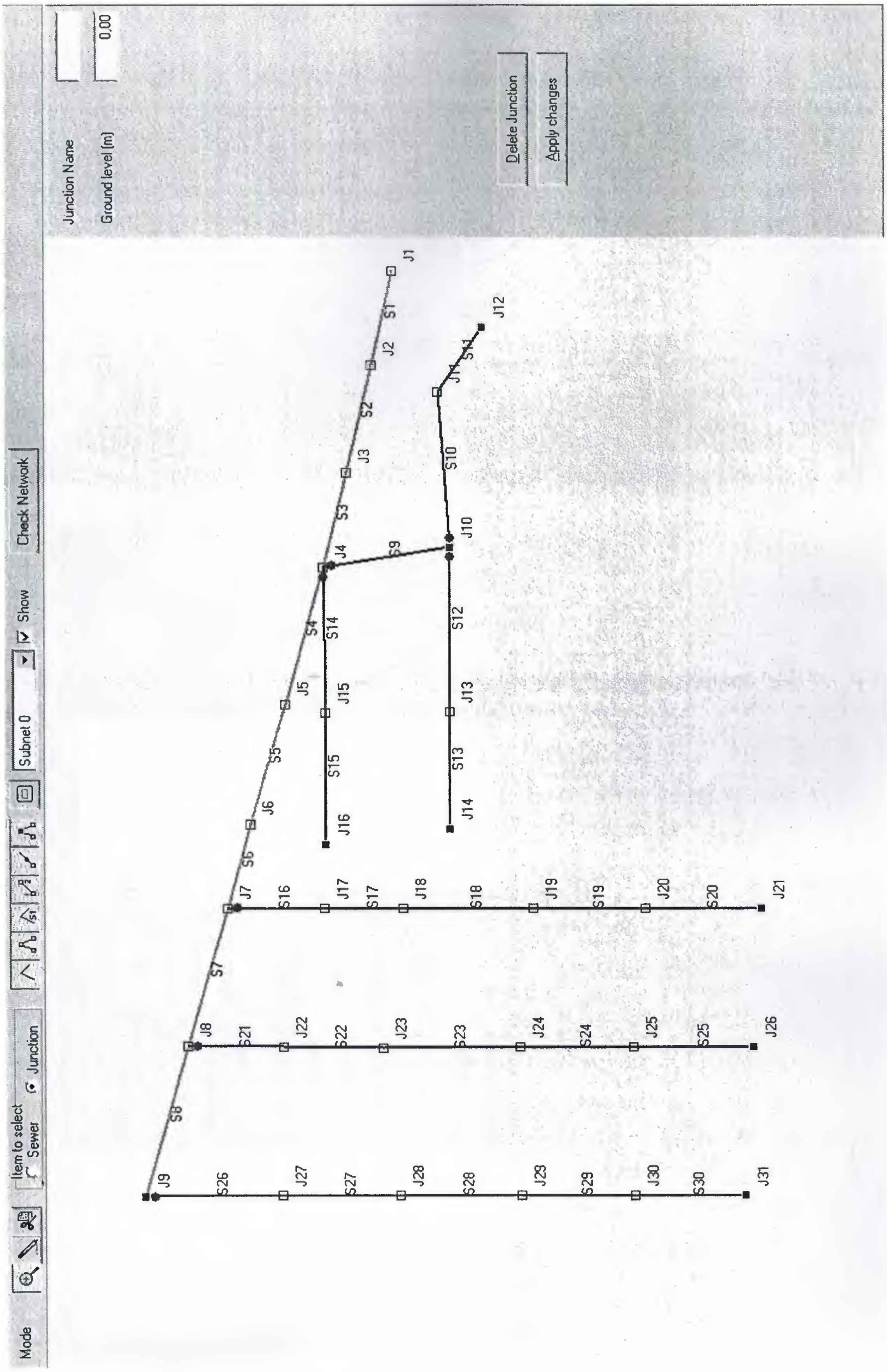

Figure 5.7 The visual editor screen, design II.

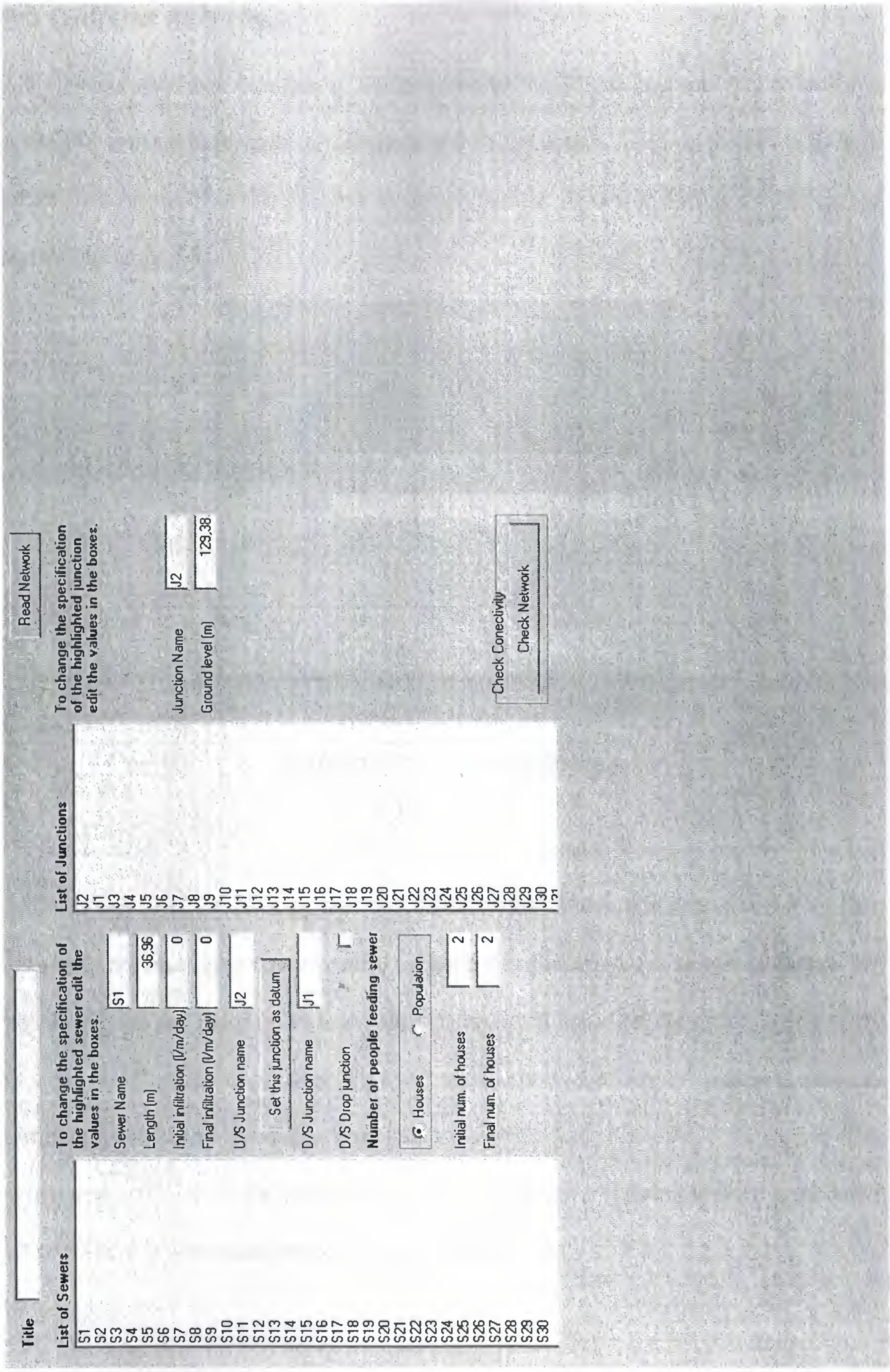

Figure 5.8 The data Entry/Edit screen, design II.

5.3 DATUM SETTING

All sub-nets need one junction to be specified as the datum junction. To do this you select the sewer whose upstream junction will be the datum. Then on the Sewer Details panel click on the "Set this junction as datum" button. This will display the dialog box shown in Figure 5.9.

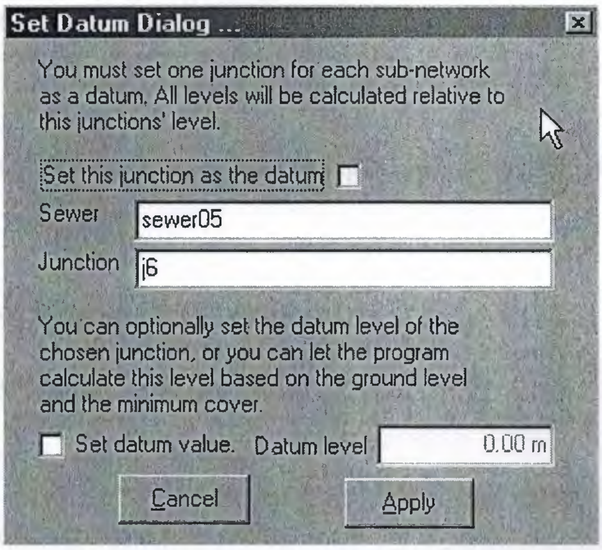

Figure 5.9 Datum setting dialog.

Click the check box to set the junction shown as the datum. You can specify the actual datum invert level by clicking the "Set datum value" check box and entering a value. Alternatively, and more conveniently, uncheck this check box (as shown in Figure 5.9) which will tell the program to calculate a datum level based on the ground level of the junction and the minimum cover. You will need to press the "Apply" button to save this information and close the dialog box.

When attempting to do the calculation, you will be given a warning if there is no datum set or if there is more than one for any particular sub-net.

5.3 DISCUSSION AND RESULTS

The software also needs some other information in order to facilitate the analysis. These information are discussed in chapter 4 which are shortly, initial water use per person; final water use per person; initial mean number of people house; final mean number of people per house; minimum self-cleansing velocity; minimum tractive tension; G-Manning's n; minimum sewer cover; return factor; peak flow factor; minimum flow; minimum diameter.

For these parameters, in order to compare design I and design II, the numbers attained are:

Initial water use per person: 50 (litres/day)

Final water use per person: 120 (litres/day)

Initial mean number of people house: 5

Final mean number of people per house: 5

Minimum self-cleansing velocity: 0.50 (m/s)

Minimum tractive tension: (N/m^2, Pa)

G-Manning's n: 0.0130

Minimum sewer cover: 0.40 (m)

Return factor: 85 (%)

Peak flow factor: 1.80

Minimum flow: 1.50 (litres/s)

Minimum diameter: 100 (mm)

All these data entrees are given in Figure 5.10. The result of the analyses is they can be taken from the result screen. The results of design I and design II are given in Figure 5.10 and Figure 5.11.

Initial water use per person (lcd) (litres/day): 50
Final water use per person (lcd) (litres/day): 120
Initial mean number of people per house: 5
Final mean number of people per house: 5
Minimum self-cleansing velocity (m/s): 0,50
Minimum tractive tension (N/m², Pa): 1,00
G-Manning's n: 0,0130
Minimum sewer cover (m): 0,40
Return factor (%): 85
Peak flow factor: 1,80
Min Vel / Min Tau (selected: Min Tau)
Minimum flow (litres/s) ☑ 1,50
Minimum diameter (mm) ☑ 100
Ground slope limiting ☑
G-Manning (selected) / CW / Escritt

Sewer	Length (m)	Total Initial flow (l/s)	Total final flow (l/s)	u/s Junct. name	d/s Junct. name	Ground level u/s (m)	Ground level d/s (m)	Groundslope	Diameter (m)	Chosen Pipe Diameter (m)	d/D Initial flow	d/D Final flow	Velocity of initial flow (m/s)	Velocity of final flow (m/s)	Depth u/s (m)	Depth d/s (m)
S1	36,51	1,500	1,500	J2	J1	125,79	125,94	-0,004	0,100	0,100	0,460	0,460	0,425	0,425	0,713	1,034
S2	58,27	1,500	1,500	J3	J2	126,61	125,79	0,014	0,100	0,100	0,340	0,340	0,637	0,637	0,711	0,713
S3	49,30	1,500	1,500	J4	J3	127,70	126,61	0,022	0,100	0,100	0,300	0,300	0,757	0,757	0,712	0,711
S4	54,84	1,500	1,500	J5	J4	129,12	127,70	0,026	0,100	0,100	0,290	0,290	0,793	0,793	0,711	0,712
S5	51,10	1,500	1,500	J6	J5	130,00	129,12	0,017	0,100	0,100	0,320	0,320	0,692	0,692	0,713	0,711
S6	45,45	1,500	1,500	J7	J6	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,500	0,713
S7	46,20	1,500	1,500	J8	J7	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,284	0,500
S8	39,82	1,500	1,500	J9	J2	126,46	125,79	0,017	0,100	0,100	0,330	0,330	0,664	0,664	0,501	0,500
S9	28,14	1,500	1,500	J10	J9	127,07	126,46	0,022	0,100	0,100	0,310	0,310	0,723	0,723	0,501	0,501
S10	26,24	1,500	1,500	J11	J10	127,51	127,07	0,017	0,100	0,100	0,330	0,330	0,664	0,664	0,500	0,501
S11	33,10	1,500	1,500	J12	J11	128,18	127,51	0,020	0,100	0,100	0,310	0,310	0,723	0,723	0,501	0,500
S12	45,27	1,500	1,500	J13	J12	129,00	128,18	0,018	0,100	0,100	0,320	0,320	0,692	0,692	0,502	0,501
S13	35,42	1,500	1,500	J14	J13	129,82	129,00	0,023	0,100	0,100	0,300	0,300	0,757	0,757	0,500	0,502
S14	31,36	1,500	1,500	J15	J9	127,05	126,46	0,019	0,100	0,100	0,320	0,320	0,692	0,692	0,500	0,499
S15	39,28	1,500	1,500	J16	J15	127,81	127,05	0,019	0,100	0,100	0,320	0,320	0,692	0,692	0,502	0,500
S16	43,41	1,500	1,500	J17	J16	128,65	127,81	0,019	0,100	0,100	0,310	0,310	0,723	0,723	0,500	0,502
S17	36,59	1,500	1,500	J18	J17	129,62	128,65	0,027	0,100	0,100	0,290	0,290	0,793	0,793	0,500	0,500
S18	37,87	1,500	1,500	J19	J6	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,500	0,677
S19	44,21	1,500	1,500	J20	J19	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,293	0,500
S20	19,94	1,500	1,500	J21	J20	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,200	0,293
S21	41,29	1,500	1,500	J22	J6	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,574	1,767
S22	39,63	1,500	1,500	J23	J22	129,60	130,00	-0,010	0,100	0,100	0,460	0,460	0,425	0,425	0,988	1,574
S23	18,88	1,500	1,500	J24	J23	129,20	129,60	-0,021	0,100	0,100	0,460	0,460	0,425	0,425	0,500	0,988
S24	42,90	1,500	1,500	J25	J7	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,500	0,701
S25	26,10	1,500	1,500	J26	J25	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,378	0,500
S26	45,65	1,500	1,500	J27	J26	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,164	0,378
S27	42,64	1,500	1,500	J28	J7	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,331	1,530
S28	40,23	1,500	1,500	J29	J28	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,142	1,331
S29	19,78	1,500	1,500	J30	J29	129,45	130,00	-0,028	0,100	0,100	0,460	0,460	0,425	0,425	0,500	1,142

Save table to CSV | Apend table to CSV | Printer Layout | Print Preview | Update Table

Figure 5.10 The results for design I.

Initial water use per person (lcd) (litres/day)	50	Minimum self-cleansing velocity (m/s)	0,50	Return factor (%)	85	Minimum flow (litres/s) ☑	1,50
Final water use per person (lcd) (litres/day)	120	Minimum tractive tension (N/m², Pa)	1,00	Peak flow factor	1,80	Minimum diameter (mm) ☑	100
Initial mean number of people per house	5	G-Manning's n	0,0130			Ground slope limiting ☑	
Final mean number of people per house	5	Minimum sewer cover (m)	0,40	○ Min Vel ◉ Min Tau		◉ G-Manning ○ CW ○ Escritt	

Sewer	Length (m)	Total Initial flow (l/s)	Total final flow (l/s)	u/s Junct. name	d/s Junct. name	Ground level u/s (m)	Ground level d/s (m)	Groundslope	Diameter (m)	Chosen Pipe Diameter (m)	d/D Initial flow	d/D Final flow	Velocity of initial flow (m/s)	Velocity of final flow (m/s)	Depth u/s (m)	Depth d/s (m)
S7	46,20	1,500	1,500	J8	J7	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,284	0,500
S8	39,82	1,500	1,500	J9	J2	126,46	125,79	0,017	0,100	0,100	0,330	0,330	0,664	0,664	0,501	0,500
S9	28,14	1,500	1,500	J10	J9	127,07	126,46	0,022	0,100	0,100	0,310	0,310	0,723	0,723	0,501	0,501
S10	26,24	1,500	1,500	J11	J10	127,51	127,07	0,017	0,100	0,100	0,330	0,330	0,664	0,664	0,500	0,501
S11	33,10	1,500	1,500	J12	J11	128,18	127,51	0,020	0,100	0,100	0,310	0,310	0,723	0,723	0,501	0,500
S12	45,27	1,500	1,500	J13	J12	129,00	128,18	0,018	0,100	0,100	0,320	0,320	0,692	0,692	0,502	0,501
S13	35,42	1,500	1,500	J14	J13	129,82	129,00	0,023	0,100	0,100	0,300	0,300	0,757	0,757	0,500	0,502
S14	31,36	1,500	1,500	J15	J9	127,05	126,46	0,019	0,100	0,100	0,320	0,320	0,692	0,692	0,500	0,499
S15	39,28	1,500	1,500	J16	J15	127,81	127,05	0,019	0,100	0,100	0,320	0,320	0,692	0,692	0,502	0,500
S16	43,41	1,500	1,500	J17	J16	128,65	127,81	0,019	0,100	0,100	0,310	0,310	0,723	0,723	0,500	0,502
S17	36,59	1,500	1,500	J18	J17	129,62	128,65	0,027	0,100	0,100	0,290	0,290	0,793	0,793	0,500	0,500
S18	37,87	1,500	1,500	J19	J6	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,500	0,677
S19	44,21	1,500	1,500	J20	J19	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,293	0,500
S20	19,94	1,500	1,500	J21	J20	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,200	0,293
S21	41,29	1,500	1,500	J22	J6	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,574	1,767
S22	39,63	1,500	1,500	J23	J22	129,60	130,00	-0,010	0,100	0,100	0,460	0,460	0,425	0,425	0,988	1,574
S23	18,88	1,500	1,500	J24	J23	129,20	129,60	-0,021	0,100	0,100	0,460	0,460	0,425	0,425	0,500	0,988
S24	42,90	1,500	1,500	J25	J7	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,500	0,701
S25	26,10	1,500	1,500	J26	J25	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,378	0,500
S26	45,65	1,500	1,500	J27	J26	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,164	0,378
S27	42,64	1,500	1,500	J28	J7	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,331	1,530
S28	40,23	1,500	1,500	J29	J28	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,142	1,331
S29	19,78	1,500	1,500	J30	J29	129,45	130,00	-0,028	0,100	0,100	0,460	0,460	0,425	0,425	0,500	1,142
S30	47,00	1,500	1,500	J31	J8	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,500	0,720
S31	45,30	1,500	1,500	J32	J31	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,288	0,500
S32	34,77	1,500	1,500	J33	J32	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,126	0,288
S33	27,52	1,500	1,500	J34	J8	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	2,061	2,190
S34	40,20	1,500	1,500	J35	J34	129,83	130,00	-0,004	0,100	0,100	0,460	0,460	0,425	0,425	1,703	2,061
S35	36,99	1,500	1,500	J36	J35	128,80	129,83	-0,028	0,100	0,100	0,460	0,460	0,425	0,425	0,500	1,703

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Figure 5.10 (Continue).

Initial water use per person (lcd) (litres/day)	50	Minimum self-cleansing velocity (m/s)	0,50	Return factor (%)	80	Minimum flow (litres/s) ☑	1,50
Final water use per person (lcd) (litres/day)	120	Minimum tractive tension (N/m², Pa)	1,00	Peak flow factor	2,40	Minimum diameter (mm) ☑	100
Initial mean number of people per house	5	G-Manning's n	0,0130			Ground slope limiting ☑	
Final mean number of people per house	5	Minimum sewer cover (m)	0,40	○ Min Vel ◉ Min Tau		○ G-Manning ○ CW ○ Escritt	

Sewer	Length (m)	Total Initial flow (l/s)	Total final flow (l/s)	u/s Junct. name	d/s Junct. name	Ground level u/s (m)	Ground level d/s (m)	Groundslope	Diameter (m)	Chosen Pipe Diameter (m)	d/D Initial flow	d/D Final flow	Velocity of initial flow (m/s)	Velocity of final flow (m/s)	Depth u/s (m)	Depth d/s (m)
S1	36,96	1,500	1,500	J2	J1	129,38	125,87	0,095	0,100	0,100	0,210	0,210	1,251	1,251	3,545	3,546
S2	35,33	1,500	1,500	J3	J2	126,96	129,38	-0,068	0,100	0,100	0,460	0,460	0,425	0,425	0,960	3,545
S3	36,41	1,500	1,500	J4	J3	127,81	126,96	0,023	0,100	0,100	0,300	0,300	0,757	0,757	0,961	0,960
S4	48,43	1,500	1,500	J5	J4	128,86	127,81	0,022	0,100	0,100	0,310	0,310	0,723	0,723	0,960	0,961
S5	46,78	1,500	1,500	J6	J5	129,70	128,86	0,018	0,100	0,100	0,320	0,320	0,692	0,692	0,958	0,960
S6	39,25	1,500	1,500	J7	J6	130,00	129,70	0,008	0,100	0,100	0,400	0,400	0,511	0,511	0,960	0,958
S7	50,99	1,500	1,500	J8	J7	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,722	0,960
S8	47,41	1,500	1,500	J9	J8	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,500	0,722
S9	47,77	1,500	1,500	J10	J4	127,45	127,81	-0,008	0,100	0,100	0,460	0,460	0,425	0,425	0,500	1,083
S10	46,51	1,500	1,500	J11	J10	126,50	127,45	-0,020	0,100	0,100	0,460	0,460	0,425	0,425	1,332	2,500
S11	34,70	1,500	1,500	J12	J11	125,83	126,50	-0,019	0,100	0,100	0,460	0,460	0,425	0,425	0,500	1,332
S12	49,81	1,500	1,500	J13	J10	128,40	127,45	0,019	0,100	0,100	0,320	0,320	0,692	0,692	0,502	0,504
S13	49,29	1,500	1,500	J14	J13	129,62	128,40	0,025	0,100	0,100	0,300	0,300	0,757	0,757	0,500	0,502
S14	48,04	1,500	1,500	J15	J4	128,80	127,81	0,021	0,100	0,100	0,310	0,310	0,723	0,723	0,499	0,498
S15	48,51	1,500	1,500	J16	J15	129,82	128,80	0,021	0,100	0,100	0,310	0,310	0,723	0,723	0,500	0,499
S16	38,10	1,500	1,500	J17	J7	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	2,123	2,301
S17	30,80	1,500	1,500	J18	J17	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,979	2,123
S18	49,27	1,500	1,500	J19	J18	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,749	1,979
S19	47,80	1,500	1,500	J20	J19	129,99	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,515	1,749
S20	48,20	1,500	1,500	J21	J20	129,20	129,99	-0,016	0,100	0,100	0,460	0,460	0,425	0,425	0,500	1,515
S21	38,18	1,500	1,500	J22	J8	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,938	2,116
S22	39,32	1,500	1,500	J23	J22	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,754	1,938
S23	50,43	1,500	1,500	J24	J23	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,518	1,754
S24	49,84	1,500	1,500	J25	J24	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,285	1,518
S25	50,27	1,500	1,500	J26	J25	129,45	130,00	-0,011	0,100	0,100	0,460	0,460	0,425	0,425	0,500	1,285
S26	48,27	1,500	1,500	J27	J9	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	2,609	2,835
S27	48,81	1,500	1,500	J28	J27	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	2,381	2,609
S28	48,74	1,500	1,500	J29	J28	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	2,153	2,381
S29	48,20	1,500	1,500	J30	J29	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,928	2,153

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Figure 5.11 The results for design II.

Initial water use per person (lcd) (litres/day)	50	Minimum self-cleansing velocity (m/s)	0,50	Return factor (%)	80	Minimum flow (litres/s) ☑	1,50
Final water use per person (lcd) (litres/day)	120	Minimum tractive tension (N/m², Pa)	1,00	Peak flow factor	2,40	Minimum diameter (mm) ☑	100
Initial mean number of people per house	5	G-Manning's n	0,0130			Ground slope limiting ☑	
Final mean number of people per house	5	Minimum sewer cover (m)	0,40	○ Min Vel ◉ Min Tau		○ G-Manning ○ CW ○ Escritt	

Sewer	Length (m)	Total Initial flow (l/s)	Total final flow (l/s)	u/s Junct. name	d/s Junct. name	Ground level u/s (m)	Ground level d/s (m)	Groundslope	Diameter (m)	Chosen Pipe Diameter (m)	d/D Initial flow	d/D Final flow	Velocity of initial flow (m/s)	Velocity of final flow (m/s)	Depth u/s (m)	Depth d/s (m)
S2	35,33	1,500	1,500	J3	J2	126,96	129,38	-0,068	0,100	0,100	0,460	0,460	0,425	0,425	0,960	3,545
S3	36,41	1,500	1,500	J4	J3	127,81	126,96	0,023	0,100	0,100	0,300	0,300	0,757	0,757	0,961	0,960
S4	48,43	1,500	1,500	J5	J4	128,86	127,81	0,022	0,100	0,100	0,310	0,310	0,723	0,723	0,960	0,961
S5	46,78	1,500	1,500	J6	J5	129,70	128,86	0,018	0,100	0,100	0,320	0,320	0,692	0,692	0,958	0,960
S6	39,25	1,500	1,500	J7	J6	130,00	129,70	0,008	0,100	0,100	0,400	0,400	0,511	0,511	0,960	0,958
S7	50,99	1,500	1,500	J8	J7	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,722	0,960
S8	47,41	1,500	1,500	J9	J8	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	0,500	0,722
S9	47,77	1,500	1,500	J10	J4	127,45	127,81	-0,008	0,100	0,100	0,460	0,460	0,425	0,425	0,500	1,083
S10	46,51	1,500	1,500	J11	J10	126,50	127,45	-0,020	0,100	0,100	0,460	0,460	0,425	0,425	1,332	2,500
S11	34,70	1,500	1,500	J12	J11	125,83	126,50	-0,019	0,100	0,100	0,460	0,460	0,425	0,425	0,500	1,332
S12	49,81	1,500	1,500	J13	J10	128,40	127,45	0,019	0,100	0,100	0,320	0,320	0,692	0,692	0,502	0,504
S13	49,29	1,500	1,500	J14	J13	129,62	128,40	0,025	0,100	0,100	0,300	0,300	0,757	0,757	0,500	0,502
S14	48,04	1,500	1,500	J15	J4	128,80	127,81	0,021	0,100	0,100	0,310	0,310	0,723	0,723	0,499	0,498
S15	48,51	1,500	1,500	J16	J15	129,82	128,80	0,021	0,100	0,100	0,310	0,310	0,723	0,723	0,500	0,499
S16	38,10	1,500	1,500	J17	J7	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	2,123	2,301
S17	30,80	1,500	1,500	J18	J17	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,979	2,123
S18	49,27	1,500	1,500	J19	J18	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,749	1,979
S19	47,80	1,500	1,500	J20	J19	129,99	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,515	1,749
S20	48,20	1,500	1,500	J21	J20	129,20	129,99	-0,016	0,100	0,100	0,460	0,460	0,425	0,425	0,500	1,515
S21	38,18	1,500	1,500	J22	J8	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,938	2,116
S22	39,32	1,500	1,500	J23	J22	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,754	1,938
S23	50,43	1,500	1,500	J24	J23	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,518	1,754
S24	49,84	1,500	1,500	J25	J24	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,285	1,518
S25	50,27	1,500	1,500	J26	J25	129,45	130,00	-0,011	0,100	0,100	0,460	0,460	0,425	0,425	0,500	1,285
S26	48,27	1,500	1,500	J27	J9	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	2,609	2,835
S27	48,81	1,500	1,500	J28	J27	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	2,381	2,609
S28	48,74	1,500	1,500	J29	J28	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	2,153	2,381
S29	48,20	1,500	1,500	J30	J29	130,00	130,00	0,000	0,100	0,100	0,460	0,460	0,425	0,425	1,928	2,153
S30	48,74	1,500	1,500	J31	J30	128,80	130,00	-0,025	0,100	0,100	0,460	0,460	0,425	0,425	0,500	1,928

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Figure 5.11 (Continue).

After the first trial it is observed that all the required restrictions are maintained. Especially the d/D ratio is checked and it is confirmed that d/D is between 0.2 and 0.8 for all the sewer connections.

The same check is done for both designs I and designs II.some other controls are carried for only design I. For example, in the menu bar, minimum diameter restriction is released and checked whether the network system will work or not. After the application of the analyses the result show that even for smaller diameter pipes, the sewerage can easily carried in the network. However, the pipe diameters that are proposed by the program are not logic. The result shows that it is necessary to use the pipe diameter which is available at the market (see Figure 5.12). This means that the minimum diameter of the network should be 100mm.

Initial water use per person (lcd) (litres/day): 50
Final water use per person (lcd) (litres/day): 120
Initial mean number of people per house: 5
Final mean number of people per house: 5
Minimum self-cleansing velocity (m/s): 0,50
Minimum tractive tension (N/m², Pa): 1,00
G-Manning's n: 0,0130
Minimum sewer cover (m): 0,40
Return factor (%): 85
Peak flow factor: 1,80
Min Vel / Min Tau (selected)
Minimum flow (litres/s) ☑ 1,50
Minimum diameter (mm) ☐ 100
Ground slope limiting ☑
G-Manning (selected) / CW / Escritt

Sewer	Length (m)	u/s Junct. name	d/s Junct. name	Ground level u/s (m)	Ground level d/s (m)	Groundslope	Gradient	Diameter (m)	Chosen Pipe Diameter (m)	d/D Initial flow	d/D Final flow	Velocity of initial flow (m/s)	Velocity of final flow (m/s)	Depth u/s (m)	Depth d/s (m)
S1	36,51	J2	J1	125,79	125,94	-0,004	0,005	0,073	0,075	0,760	0,760	0,416	0,416	0,688	1,009
S2	58,27	J3	J2	126,61	125,79	0,014	0,014	0,059	0,075	0,520	0,520	0,646	0,646	0,686	0,688
S3	49,30	J4	J3	127,70	126,61	0,022	0,022	0,055	0,075	0,453	0,453	0,770	0,770	0,687	0,686
S4	54,84	J5	J4	129,12	127,70	0,026	0,026	0,053	0,075	0,440	0,440	0,801	0,801	0,686	0,687
S5	51,10	J6	J5	130,00	129,12	0,017	0,017	0,057	0,075	0,493	0,493	0,691	0,691	0,688	0,686
S6	45,45	J7	J6	130,00	130,00	0,000	0,005	0,073	0,075	0,760	0,760	0,416	0,416	0,475	0,688
S7	46,20	J8	J7	130,00	130,00	0,000	0,005	0,073	0,075	0,760	0,760	0,416	0,416	0,259	0,475
S8	39,82	J9	J2	126,46	125,79	0,017	0,017	0,058	0,075	0,493	0,493	0,691	0,691	0,476	0,475
S9	28,14	J10	J9	127,07	126,46	0,022	0,022	0,055	0,075	0,467	0,467	0,742	0,742	0,476	0,476
S10	26,24	J11	J10	127,51	127,07	0,017	0,017	0,058	0,075	0,493	0,493	0,691	0,691	0,475	0,476
S11	33,10	J12	J11	128,18	127,51	0,020	0,020	0,056	0,075	0,467	0,467	0,742	0,742	0,476	0,475
S12	45,27	J13	J12	129,00	128,18	0,018	0,018	0,057	0,075	0,480	0,480	0,715	0,715	0,477	0,476
S13	35,42	J14	J13	129,82	129,00	0,023	0,023	0,054	0,075	0,453	0,453	0,770	0,770	0,475	0,477
S14	31,36	J15	J9	127,05	126,46	0,019	0,019	0,056	0,075	0,480	0,480	0,715	0,715	0,475	0,474
S15	39,28	J16	J15	127,81	127,05	0,019	0,019	0,056	0,075	0,480	0,480	0,715	0,715	0,477	0,475
S16	43,41	J17	J16	128,65	127,81	0,019	0,019	0,056	0,075	0,480	0,480	0,715	0,715	0,475	0,477
S17	36,59	J18	J17	129,62	128,65	0,027	0,026	0,053	0,075	0,440	0,440	0,801	0,801	0,475	0,475
S18	37,87	J19	J6	130,00	130,00	0,000	0,005	0,073	0,075	0,760	0,760	0,416	0,416	0,475	0,652
S19	44,21	J20	J19	130,00	130,00	0,000	0,005	0,073	0,075	0,760	0,760	0,416	0,416	0,268	0,475
S20	19,94	J21	J20	130,00	130,00	0,000	0,005	0,073	0,075	0,760	0,760	0,416	0,416	0,175	0,268
S21	41,29	J22	J6	130,00	130,00	0,000	0,005	0,073	0,075	0,760	0,760	0,416	0,416	1,549	1,742
S22	39,63	J23	J22	129,60	130,00	-0,010	0,005	0,073	0,075	0,760	0,760	0,416	0,416	0,963	1,549
S23	18,88	J24	J23	129,20	129,60	-0,021	0,005	0,073	0,075	0,760	0,760	0,416	0,416	0,475	0,963
S24	42,90	J25	J7	130,00	130,00	0,000	0,005	0,073	0,075	0,760	0,760	0,416	0,416	0,475	0,676
S25	26,10	J26	J25	130,00	130,00	0,000	0,005	0,073	0,075	0,760	0,760	0,416	0,416	0,353	0,475
S26	45,65	J27	J26	130,00	130,00	0,000	0,005	0,073	0,075	0,760	0,760	0,416	0,416	0,139	0,353
S27	42,64	J28	J7	130,00	130,00	0,000	0,005	0,073	0,075	0,760	0,760	0,416	0,416	1,306	1,505
S28	40,23	J29	J28	130,00	130,00	0,000	0,005	0,073	0,075	0,760	0,760	0,416	0,416	1,117	1,306
S29	19,78	J30	J29	129,45	130,00	-0,028	0,005	0,073	0,075	0,760	0,760	0,416	0,416	0,475	1,117

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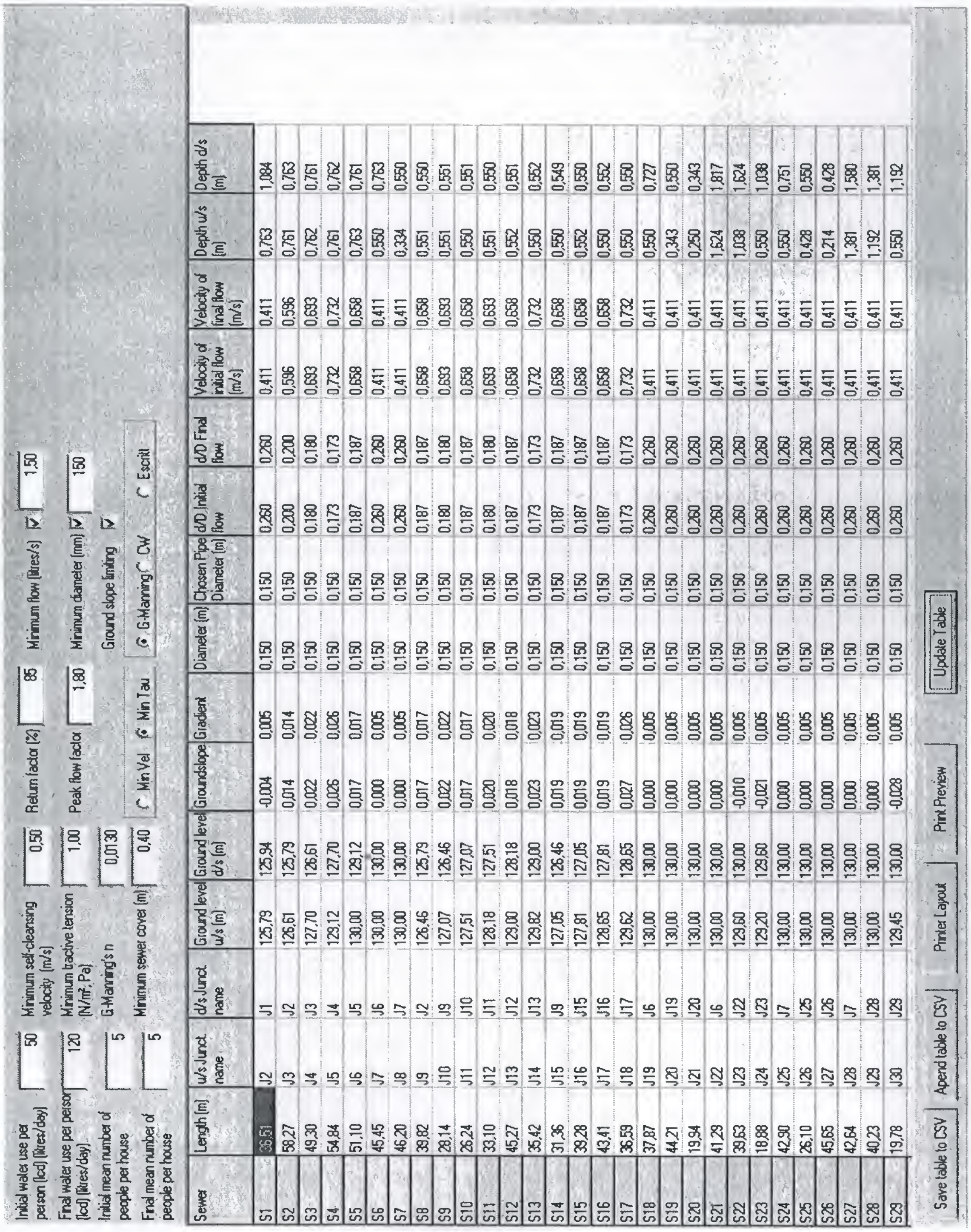
Figure 5.12 The result without ticking the minimum diameter check box.

The next step is to check the diameter of 150mm is suitable or not. On this purpose the minimum diameter of the pipes are set to 150mm.The model is run and the results are analyzed. According to the results it is observed that the first check which is the d/D ratio is not satisfied (see Figure 5.13).



Sewer	Length (m)	u/s Junct. name	d/s Junct. name	Ground level u/s (m)	Ground level d/s (m)	Groundslope	Gradient	Diameter (m)	Chosen Pipe Diameter (m)	d/D Initial flow	d/D Final flow	Velocity of initial flow (m/s)	Velocity of final flow (m/s)	Depth u/s (m)	Depth d/s (m)
S1	36,51	J2	J1	125,79	125,94	-0,004	0,005	0,150	0,150	0,260	0,260	0,411	0,411	0,763	1,084
S2	58,27	J3	J2	126,61	125,79	0,014	0,014	0,150	0,150	0,200	0,200	0,596	0,596	0,761	0,763
S3	49,30	J4	J3	127,70	126,61	0,022	0,022	0,150	0,150	0,180	0,180	0,693	0,693	0,762	0,761
S4	54,84	J5	J4	129,12	127,70	0,026	0,026	0,150	0,150	0,173	0,173	0,732	0,732	0,761	0,762
S5	51,10	J6	J5	130,00	129,12	0,017	0,017	0,150	0,150	0,187	0,187	0,658	0,658	0,763	0,761
S6	45,45	J7	J6	130,00	130,00	0,000	0,005	0,150	0,150	0,260	0,260	0,411	0,411	0,550	0,763
S7	46,20	J8	J7	130,00	130,00	0,000	0,005	0,150	0,150	0,260	0,260	0,411	0,411	0,334	0,550
S8	39,82	J9	J2	126,46	125,79	0,017	0,017	0,150	0,150	0,187	0,187	0,658	0,658	0,551	0,550
S9	28,14	J10	J9	127,07	126,46	0,022	0,022	0,150	0,150	0,180	0,180	0,693	0,693	0,551	0,551
S10	26,24	J11	J10	127,51	127,07	0,017	0,017	0,150	0,150	0,187	0,187	0,658	0,658	0,550	0,551
S11	33,10	J12	J11	128,18	127,51	0,020	0,020	0,150	0,150	0,180	0,180	0,693	0,693	0,551	0,550
S12	45,27	J13	J12	129,00	128,18	0,018	0,018	0,150	0,150	0,187	0,187	0,658	0,658	0,552	0,551
S13	35,42	J14	J13	129,82	129,00	0,023	0,023	0,150	0,150	0,173	0,173	0,732	0,732	0,550	0,552
S14	31,36	J15	J9	127,05	126,46	0,019	0,019	0,150	0,150	0,187	0,187	0,658	0,658	0,550	0,549
S15	39,28	J16	J15	127,81	127,05	0,019	0,019	0,150	0,150	0,187	0,187	0,658	0,658	0,552	0,550
S16	43,41	J17	J16	128,65	127,81	0,019	0,019	0,150	0,150	0,187	0,187	0,658	0,658	0,550	0,552
S17	36,59	J18	J17	129,62	128,65	0,027	0,026	0,150	0,150	0,173	0,173	0,732	0,732	0,550	0,550
S18	37,87	J19	J6	130,00	130,00	0,000	0,005	0,150	0,150	0,260	0,260	0,411	0,411	0,550	0,727
S19	44,21	J20	J19	130,00	130,00	0,000	0,005	0,150	0,150	0,260	0,260	0,411	0,411	0,343	0,550
S20	19,94	J21	J20	130,00	130,00	0,000	0,005	0,150	0,150	0,260	0,260	0,411	0,411	0,250	0,343
S21	41,29	J22	J6	130,00	130,00	0,000	0,005	0,150	0,150	0,260	0,260	0,411	0,411	1,624	1,817
S22	39,63	J23	J22	129,60	130,00	-0,010	0,005	0,150	0,150	0,260	0,260	0,411	0,411	1,038	1,624
S23	18,88	J24	J23	129,20	129,60	-0,021	0,005	0,150	0,150	0,260	0,260	0,411	0,411	0,550	1,038
S24	42,90	J25	J7	130,00	130,00	0,000	0,005	0,150	0,150	0,260	0,260	0,411	0,411	0,550	0,751
S25	26,10	J26	J25	130,00	130,00	0,000	0,005	0,150	0,150	0,260	0,260	0,411	0,411	0,428	0,550
S26	45,65	J27	J26	130,00	130,00	0,000	0,005	0,150	0,150	0,260	0,260	0,411	0,411	0,214	0,428
S27	42,64	J28	J7	130,00	130,00	0,000	0,005	0,150	0,150	0,260	0,260	0,411	0,411	1,381	1,580
S28	40,23	J29	J28	130,00	130,00	0,000	0,005	0,150	0,150	0,260	0,260	0,411	0,411	1,192	1,381
S29	19,78	J30	J29	129,45	130,00	-0,028	0,005	0,150	0,150	0,260	0,260	0,411	0,411	0,550	1,192

Figure 5.13 The result for pipe diameter 150mm.

The calculation of the volume of excavation for design I and design II are given in Figure 5.14 and Figure 5.15 respectively.

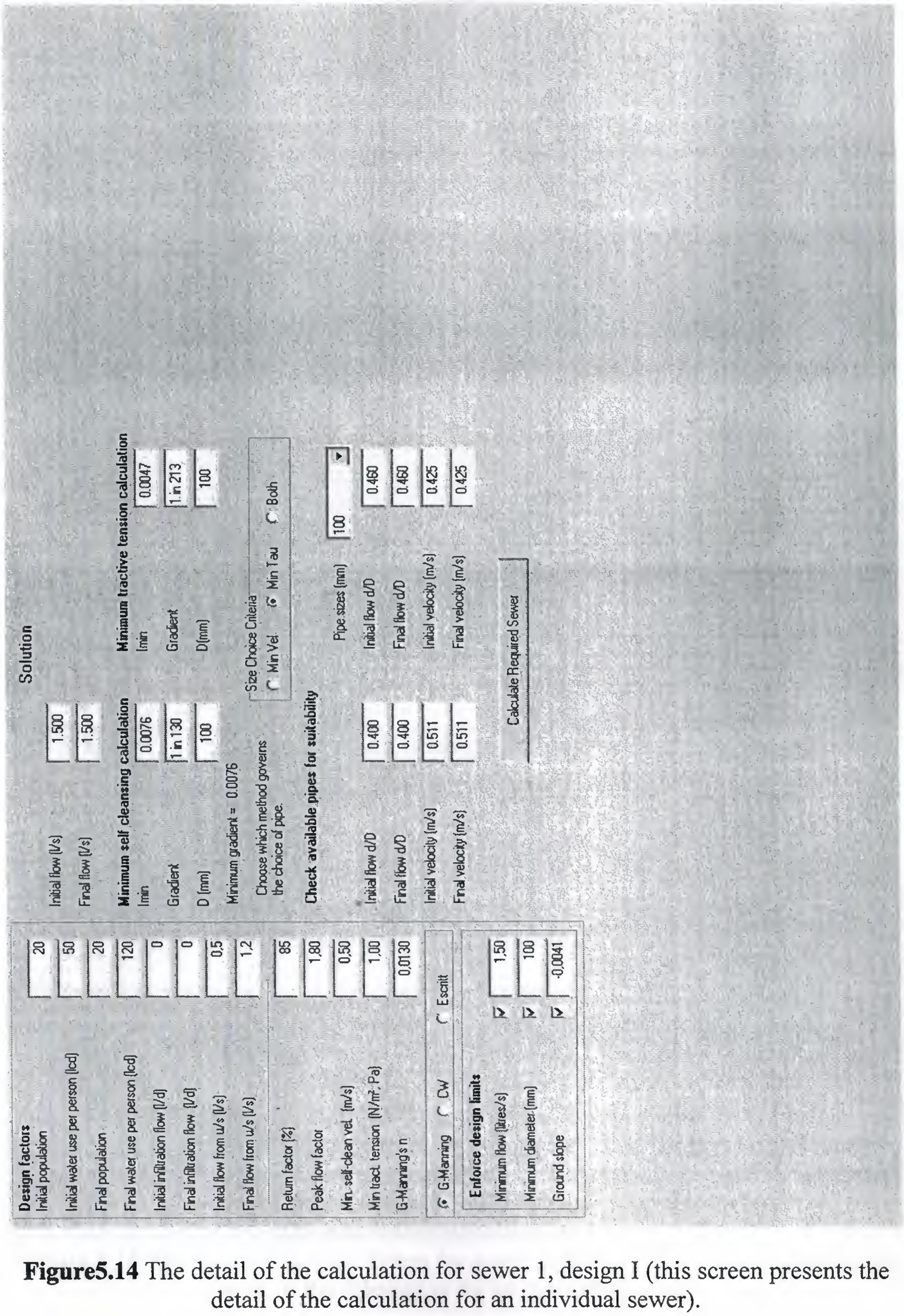


Figure5.14 The detail of the calculation for sewer 1, design I (this screen presents the detail of the calculation for an individual sewer).

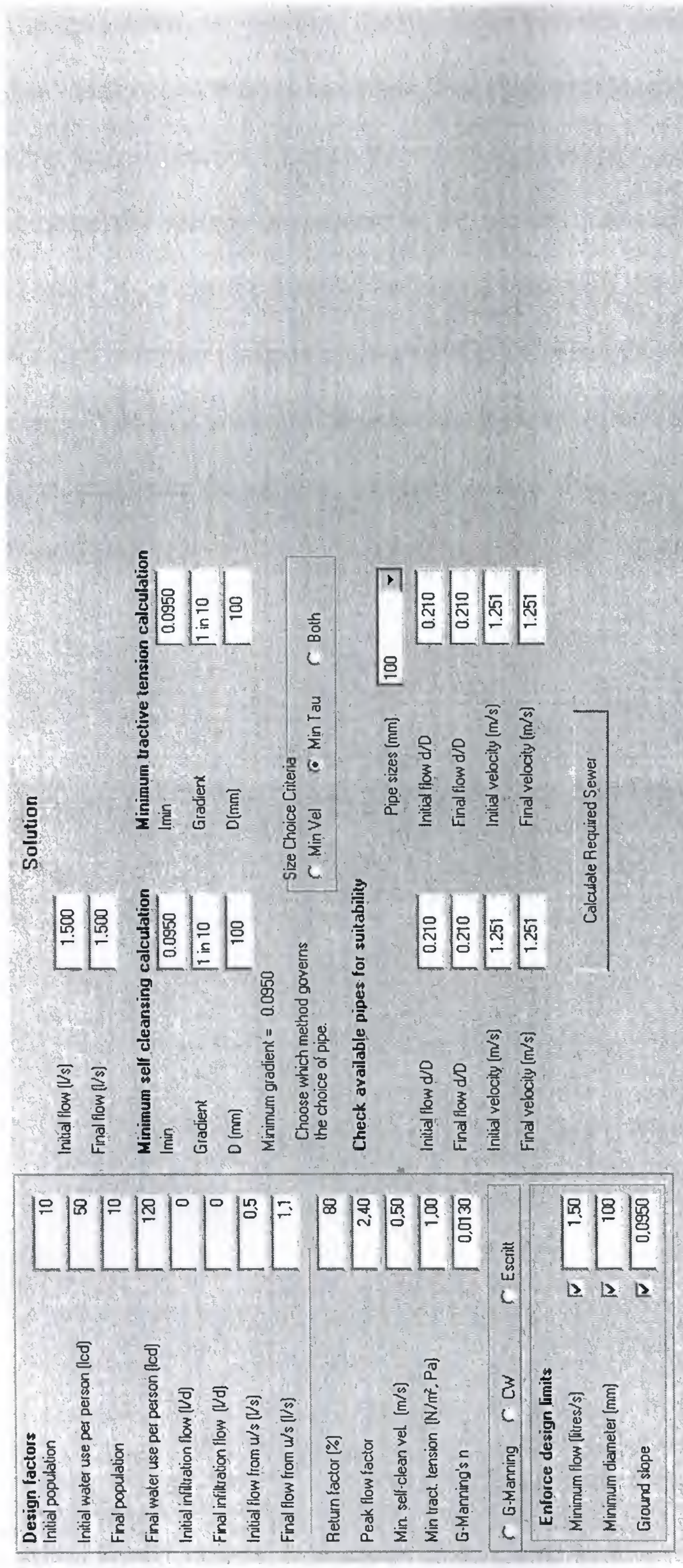


Figure 5.15 The detail of the calculation for sewer 1; design II (this screen presents the detail of the calculation for an individual sewer).

The main discussion between the two sewer network designs, design I and design II, is that which one of them is economic. Since the total length of pipe line is approximately same, only difference between the two designs is the volume of excavation. In order to compare the volume of excavation we use the information given in Figure 5.9, and Figure 5.10, where the depth of the pipe is subtracted from the ground level.

Another software program is then used to calculate the volume of excavation for each design. The area is calculated under the logic of Area = length × height and volume is calculated under the logic of, Volume = area × width (see Figure 5.16 and Figure 5.17).

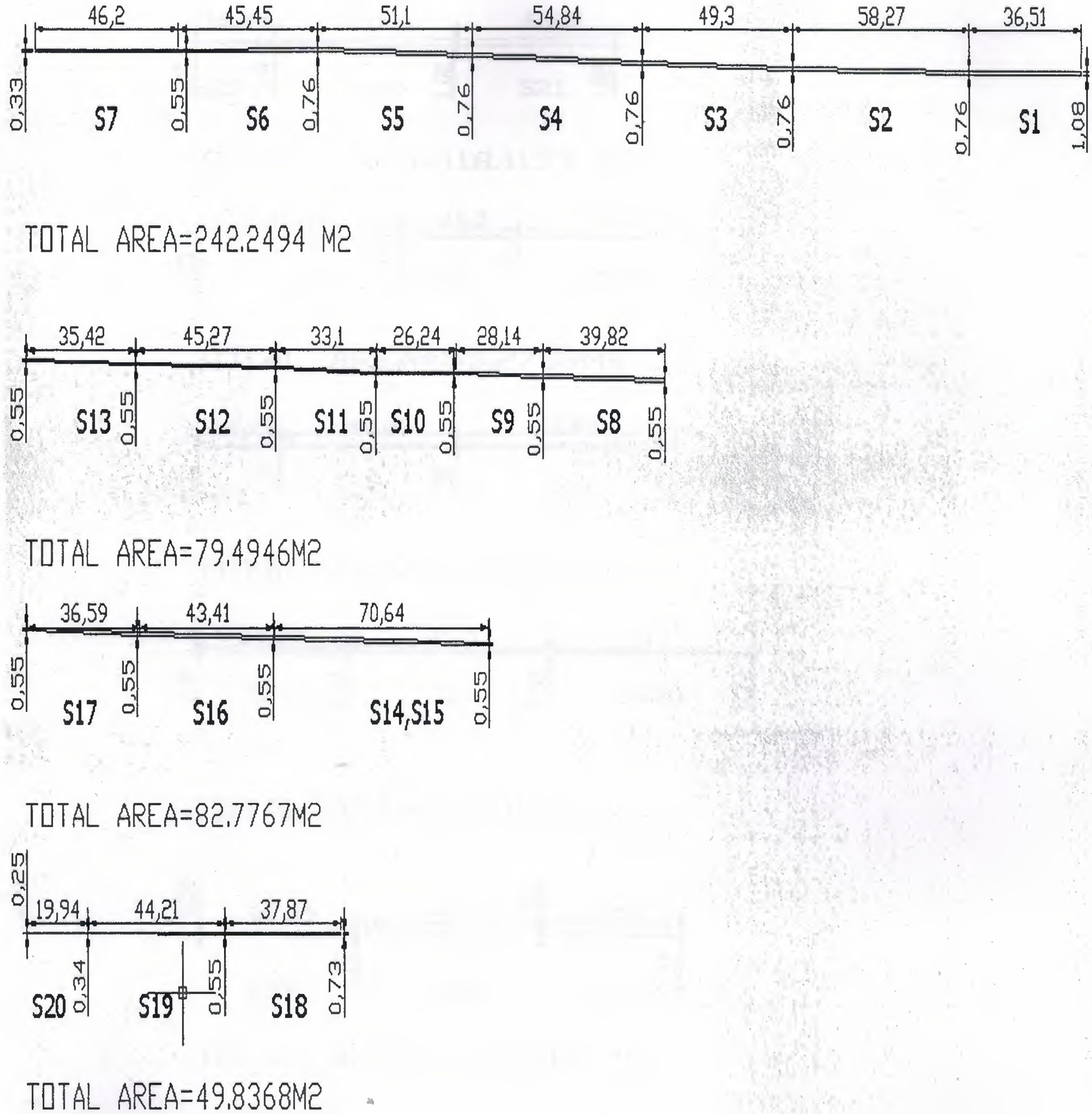


Figure 5.16 The area for design I.

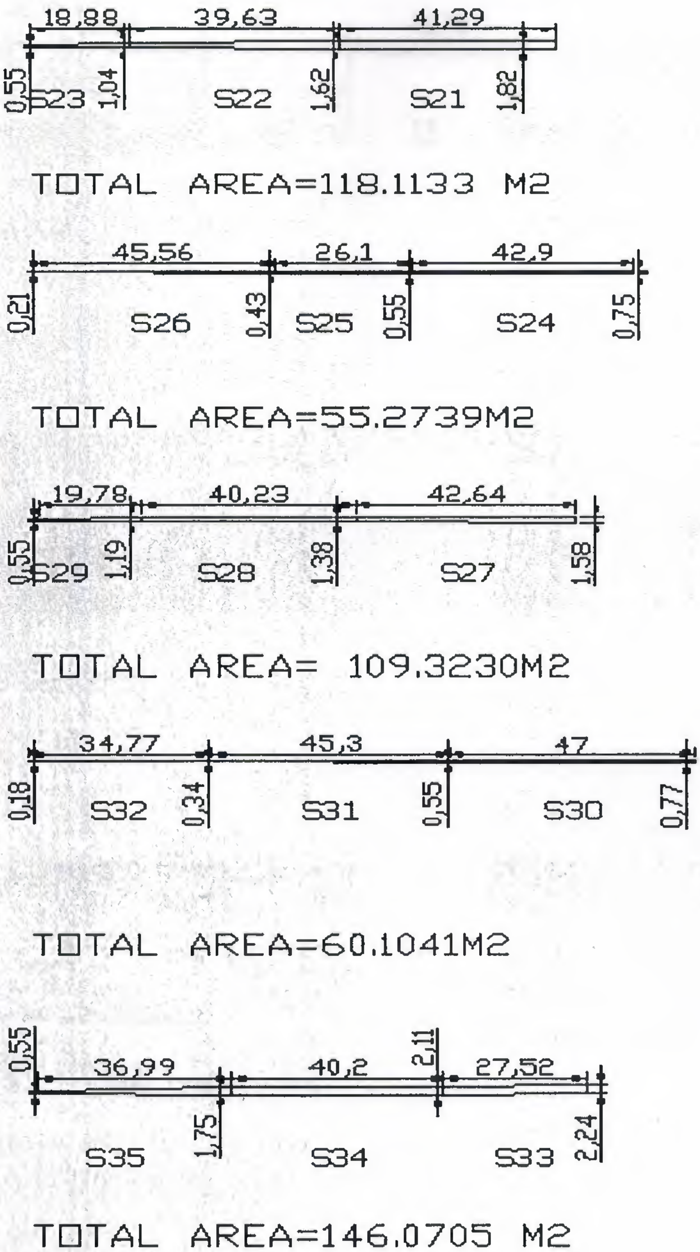
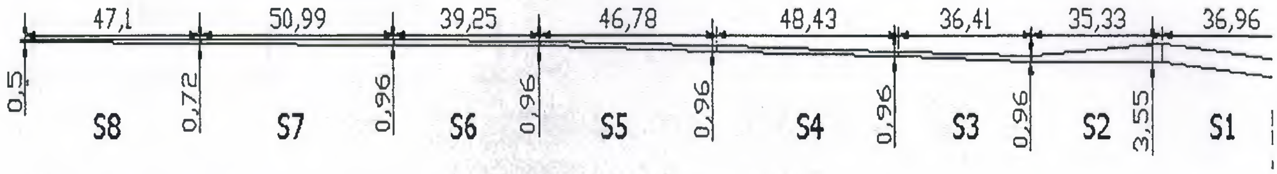


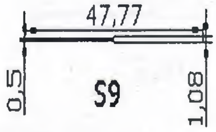
Figure 5.16 (Continue)

The all total area for design I = 943.2423m^2.

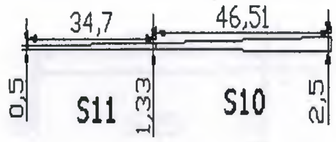
The amount of excavation = 943.24m^2 $\times$ 0.8m =754.6m^3.



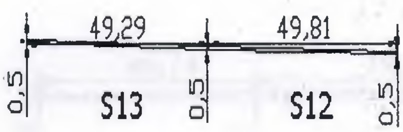
TOTAL AREA=446.4598 M2



TOTAL AREA=37.81 M2



TOTAL AREA=120.8942 M2



TOTAL AREA=49.7487 M2

Figure 5.17 The area for design II.

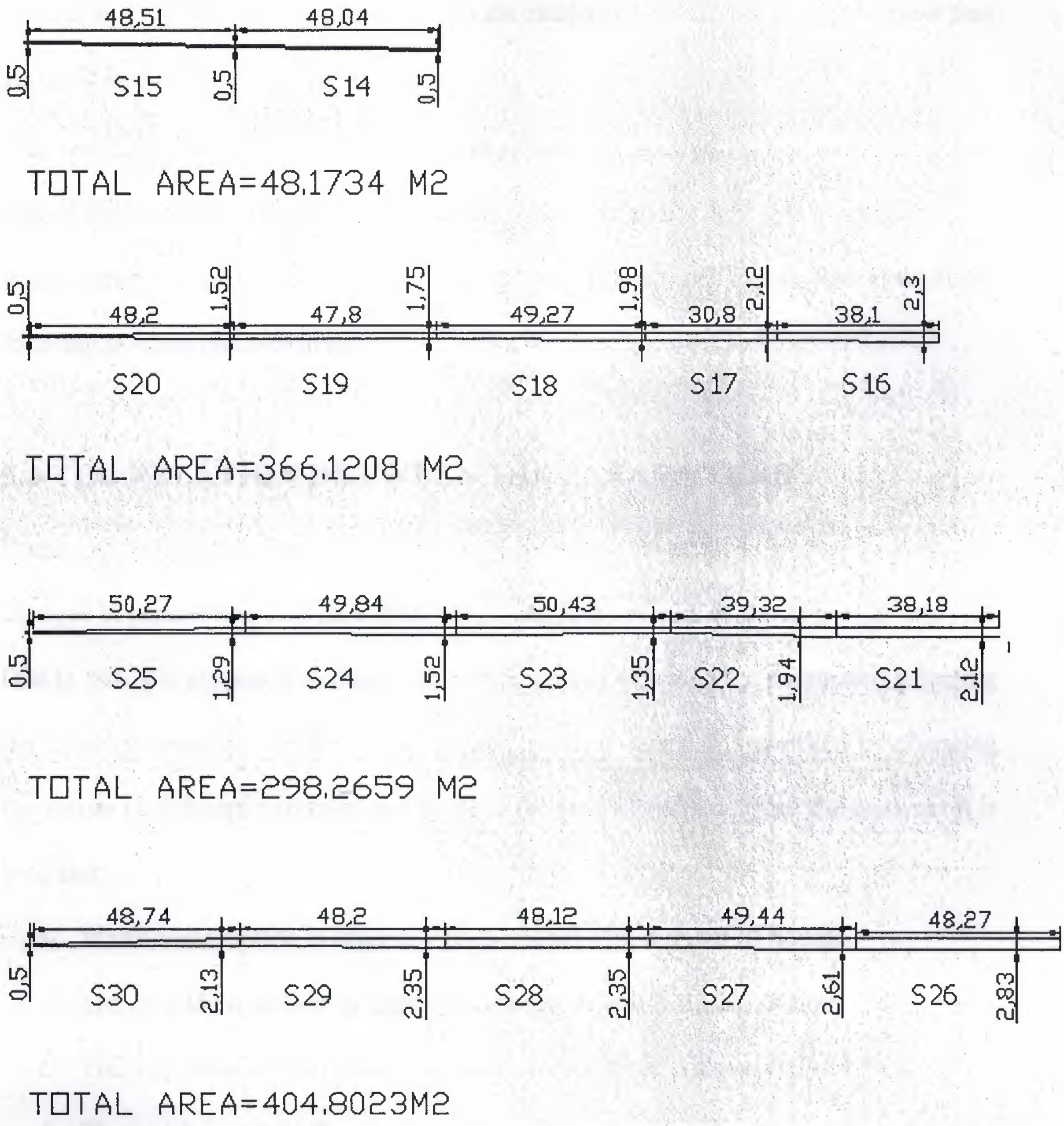


Figure 5.17 (Continue).

The all total area for design II = $1772.3m^2$.

The amount of excavation = $1772.3m^2 \times 0.8m = 1417.84m^3$.

According to the results it is observed that the excavation volume of design I is less than design II by 87.89 %.

This result mentions that shorter backyard network system will support the economic way of sewer design. This is mainly because of the slop of the network. Since the slop is nearly same for all the connections, the shorter distances will need less excavation, resulting in economic solutions.

5.4 THE RELATION BETWEEN THE PARAMETERS

During all the studies that is carried over in previous chapters, it is observed that the changes in parameters, like pipe diameter, changes the results of the sewerage analysis. Due to this, it is proposed to check the proportionality effect of the parameters affecting the sewerage analysis. To do so the feasible result of design I is checked by changing the values of different variables and keeping the others constant. After the analyses it is seen that:

a- Water consumption is inversely proportional with number of houses.

b- The slop of the sewers is directly proportional with velocity of flow.

c- The roughness of the pipes is inversely proportional with velocity of flow.

d- The roughnesses of the pipes are inversely proportional with number of houses connected to the sewer.

In Figure 5.18 and figure 5.19 it is studied to check the changes of water consumption with respect to number of houses connected to sewer. In Figure 5.18 the pipe diameter is 100mm where is in Figure 5.19 it is 150mm. The result depicted that for the constant water consumption, any increment for the pipe diameter from 100mm to 150mm will increase the number of houses Quadric.

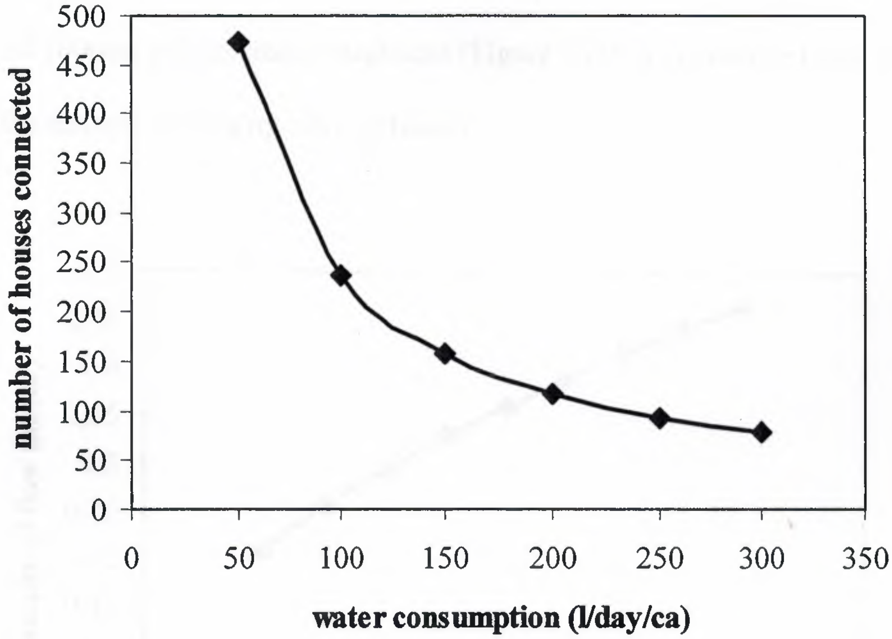


Figure 5.18 The change of number of houses connected to sewer with respect to water consumption, for a sewer diameter of 100mm.

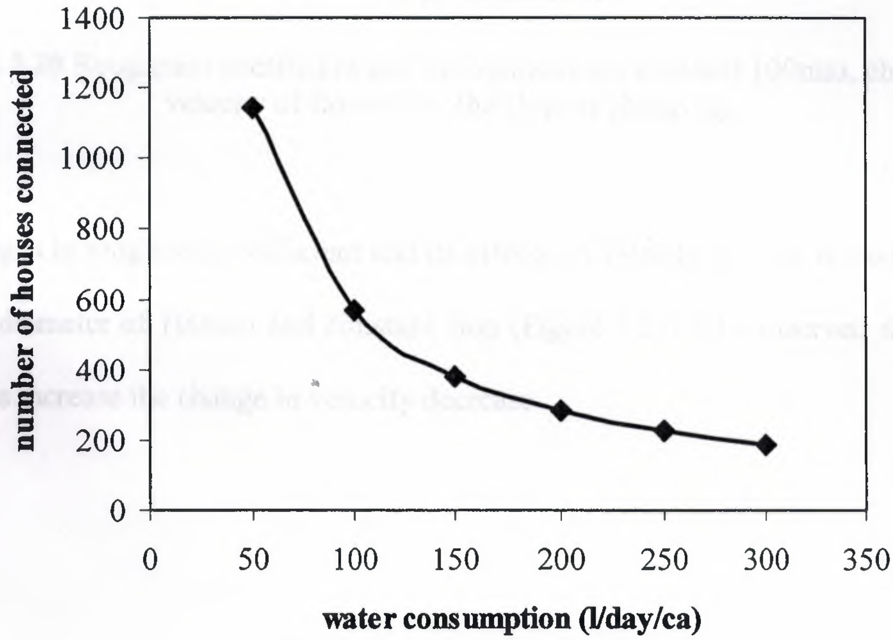


Figure 5.19 The change of number of houses connected to sewer with respect to water consumption, for a sewer diameter of 150mm.

The changes in slop and its effects on velocity of flow is studies under constant diameter of 100mm and constant roughness (Figure 5.20).It is observed that as the slope increase the change in velocity also increases.

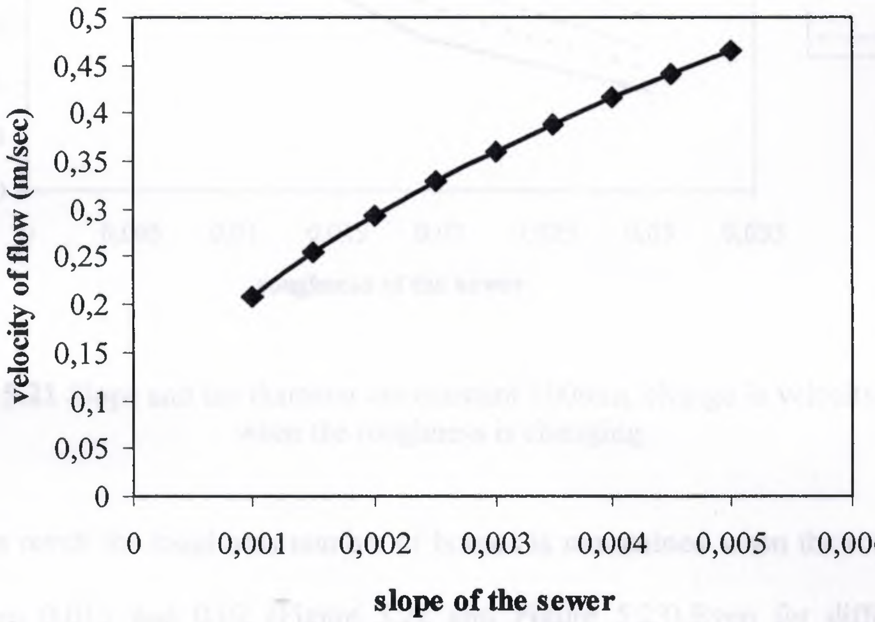


Figure 5.20 Roughness coefficient and the diameter are constant 100mm, change in velocity of flow when the slope is changing.

The changes in roughness coefficient and its effects on velocity of flow is studies under constant diameter of 100mm and constant slop (Figure 5.21).It is observed that as the roughness increase the change in velocity decrease.

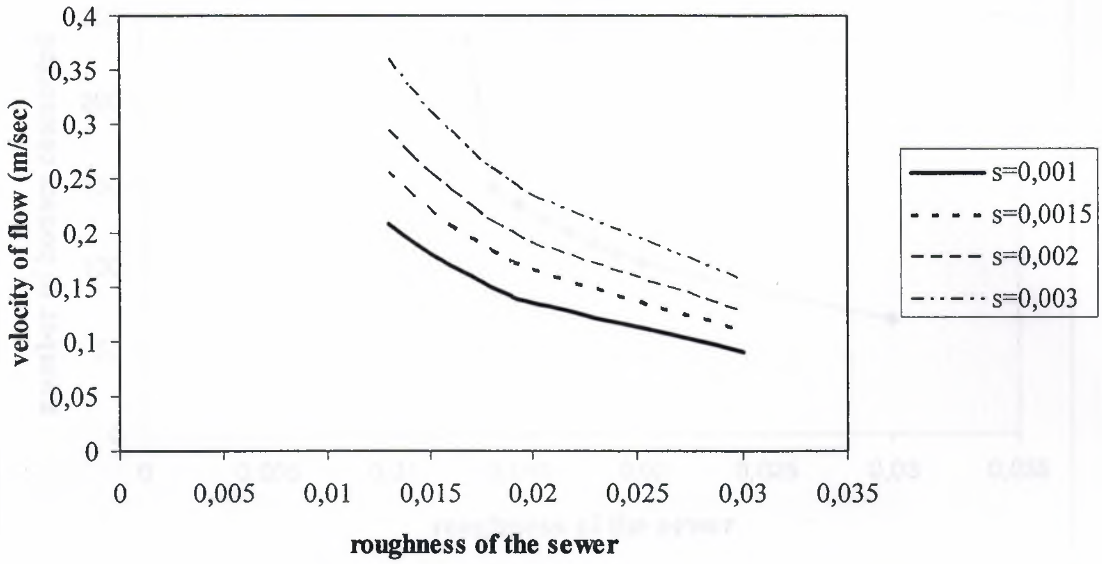


Figure 5.21 Slope and the diameter are constant 100mm, change in velocity of flow when the roughness is changing.

A feasible result for roughness-number of houses is maintained when the roughness is in between 0.013 and 0.02 (Figure 5.22 and Figure 5.23).Even for different pipe diameters it is observed that if roughness shows any deficiency from the limit of 0.013 and 0.02, the number of houses changes defiantly.

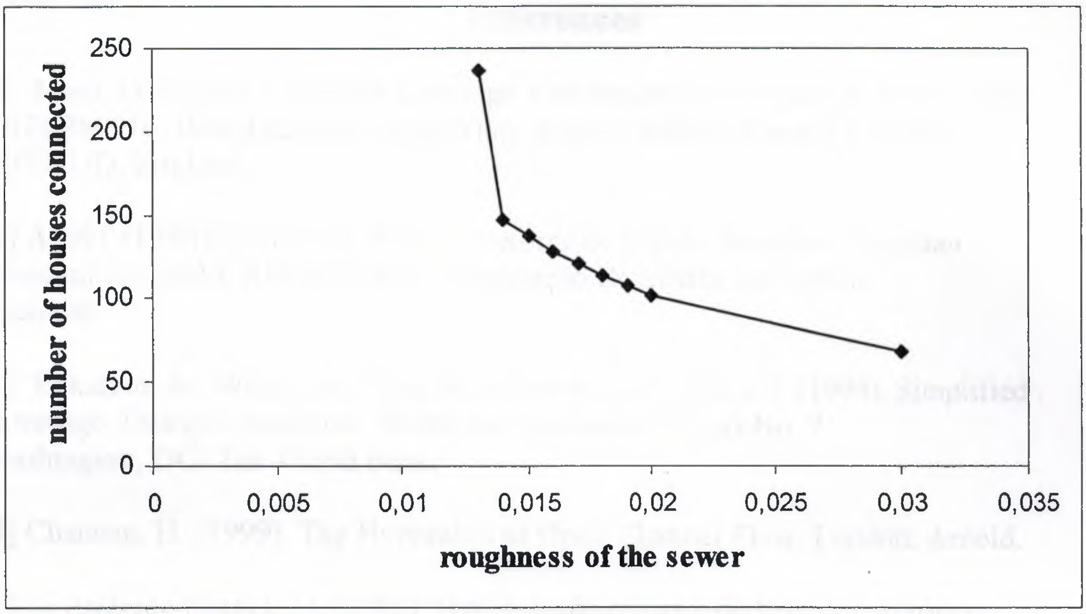


Figure 5.22 The change of number of houses connected to sewer with respect to roughness of the sewer, for a sewer diameter of 100mm.

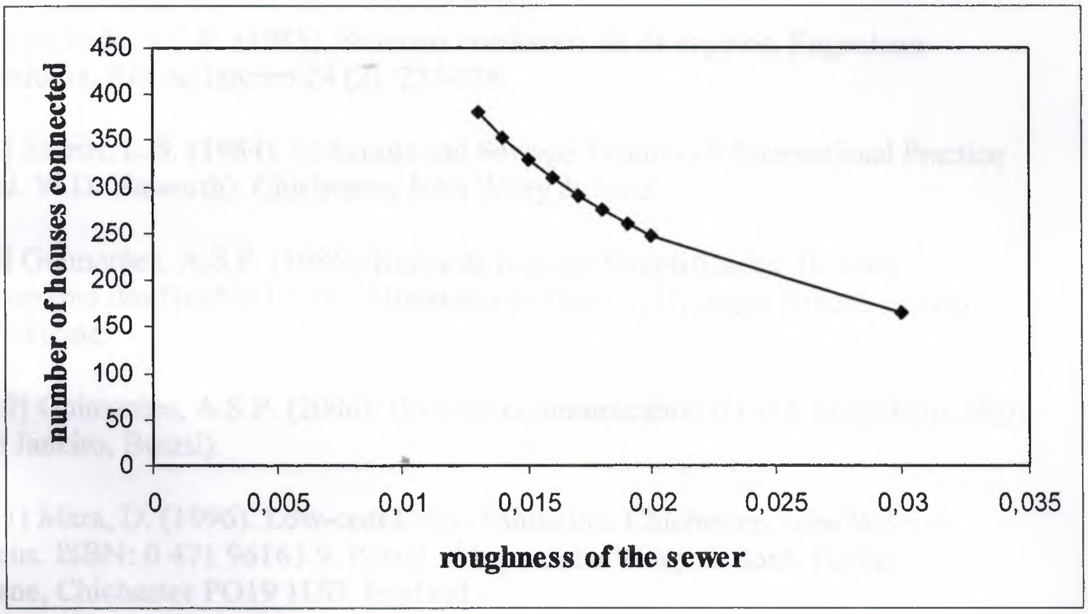


Figure 5.23 The change of number of houses connected to sewer with respect to roughness of the sewer, for a sewer diameter of 150mm.

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CONCLUSIONS

The project has investigated the theory of the hydraulic design of simplified sewerage systems. In addition, the standards associated with the condominial designs and PC-based simplified sewer design program led to savings in the length and diameter of pipes for water and sewerage. Savings in the volume of soil excavation as a result of shallower trenches. Simplified sewerage systems is much more important because its less pipeline is needed, the design requires a narrower diameter pipeline. And, pipes can be buried at a shallower depth because there is no need to protect them from the weight of passing vehicles.

Chapter 1. of the project about simplified sewerage systems introduction and the effects of sewerage systems dealing with health and environmental concerns.

Chapter 2. has investigated the theory hydraulic design of simplified sewerage system. The results of a simplified sewer design are presented, with a comparison of designs based on the Gauckler- Manning, Colebrook-White and Escritt equations.

Chapter 3. The planning and design process. The theory introduced in chapter 2 allows a sewer system to be analyzed such that, sewer diameters and gradients can be determined.

Chapter 4. Pc-based Simplified Sewer Design. Simplified sewer design is used to generate possible outcomes of the method by means of a computer program.

Chapter5. The case study about the simplify sewer method is applied to the Taskinkoy region, which is one of the most populated area of capital lefkosa.