



Fabricated fittings are hydraulically superior.

Hydraulic Design of Storm Sewers

125

CHAPTER 5

The hydraulic design of a sewer system may have to take into account the effect of backwater (the limiting effect on flows that a downstream sewer has on upstream sewers), surcharging, inlet capacity and all energy losses in the system. Whether each, or all, of these factors have to be considered depends on the complexity of the sewer system and the objectives of the analysis (i.e., is the sizing of the system preliminary or final?). Furthermore, the degree of analysis will also depend on the potential impact should the sewer system capacity be exceeded. For example, would surcharging result in damages to private property due to the foundation drains being connected to the system or is the depth of flooding on a roadway important because emergency vehicles depend on safe access along the street. By defining the above factors the user may then select the level of analysis which is required.

This section will outline two methods using hand calculations. Both methods assume that all flows enter the sewer system, i.e., that the inlet capacity of the system is not a limiting factor. In addition, a listing of various computer models which may be used in the analysis or design of sewer systems is provided.

Flow charts and nomographs such as those presented in Chapter 4 provide quick answers for the friction head losses in a given run of straight conduit between structures (manholes, junctions). These design aids do not consider the additional head losses associated with other structures and appurtenances common in sewer systems.

In most instances, when designing with common friction flow formulae such as the Manning equation, the hydraulic grade is assumed to be equal to the pipe slope at an elevation equal to the crown of the pipe. Consideration must therefore also be given to the changes in hydraulic grade line due to pressure changes, elevation changes, manholes and junctions. The design should then not only be based on the pipe slope, but on the hydraulic grade line.

A comprehensive storm sewer design must therefore proceed on the basis of one run of conduit or channel at a time, working methodically through the system. Only in this way can the free flow conditions be known and the hydraulic grade controlled, thus assuring performance of the system.

Making such an analysis requires backwater calculations for each run of conduit. This is a detailed process which is demonstrated on the following pages. However, it is recognized that a reasonable, conservative "estimate" or "shortcut" will sometimes be required. This can be done and is also demonstrated on pages 134 through 138.

When using the backwater curve approach the designer should first establish the type of flow (sub-critical or supercritical) in order to determine the direction his calculations are to proceed.

- Super critical flow - designer works downstream with flow.
- Sub-critical flow - designer works against the flow.
- Hydraulic jump may form if there is super and sub-critical flow in the same sewer.

BACKWATER ANALYSIS

Given is a flow profile of a storm drainage system (see Figures 5.1 and

5.2) where the hydraulic grade is set at the crown of the outlet pipe. Hydrological computations have been made and preliminary design for the initial pipe sizing has been completed.

In order to demonstrate the significance of form losses in sewer design, a backwater calculation will be performed in this example with helical corrugated steel pipe.

Solution

- 1) Draw a plan and surface profile of trunk storm sewer.
- 2) Design discharges, Q are known; Areas, A are known; Diameters of pipe, D have been calculated in preliminary design.
- 3) Calculate the first section of sewer line. Note: Normal depth is greater than critical depth, $y_n > y_c$; therefore, calculations to begin at outfall working upstream. At "point of control" set design conditions on profile and calculations sheet:

Station 0 + 00 (outfall)

| | | |
|---------------------------|--------------------------------|-----|
| Design discharge | $Q = 7.0 \text{ m}^3/\text{s}$ | (9) |
| Invert of pipe | $= 28.2 \text{ m}$ | (2) |
| Diameter | $D = 1800 \text{ mm}$ | (3) |
| Hydraulic grade elevation | $H.G. = 30 \text{ m}$ | (4) |
| Area of pipe | $A = 2.54 \text{ m}^2$ | (6) |
| Velocity $= \frac{Q}{A}$ | $V = 2.8 \text{ m/s}$ | (8) |

Note: (1) Numbers in parentheses refer to the columns on Table 5.2.

Compute:

a) 'K' value (7): $K = (2g) n^2$ (Derived from Manning-Chezy Formula)

$$b) 'S_f' \text{ value (12): } S_f = \left[K \frac{V^2}{2g} \right] / R^{4/3}$$

The friction slope (S_f) may also be estimated from Table 5.1 for a given diameter of pipe and with a known 'n' value for the expected flow Q .

S_f (12) is a "point slope" at each station set forth by the designer. Therefore, the friction slope (Avg. S_f) (13) for each reach of pipe L (14), is the average of the two point slopes S_f being considered.

$$c) \text{ Velocity Head (10): } H_v = \frac{V^2}{2g}$$

d) Energy grade point, E. G. (11) is equal to H. G. (4) plus the velocity head (10).

e) Friction loss (15): Multiply Avg. S_f (13) by length of sewer section, L (14) = H_f (15).

f) Calculate energy losses: H_b , H_j , H_m , H_t , using formulae in text.

g) Compute new H. G. (4) by adding all energy loss columns, (15) thru (19) to previous H. G.

Note: If sewer system is designed under pressure (surcharging) then energy losses must be added (or subtracted, depending on whether you are working upstream or downstream) to the energy grade line, E. G.

h) Set new E. G. (20) equal to E. G. (11)

Table 5.1 Energy-loss solution by Manning's formula for pipe flowing full

| Diameter (mm) | Area (m ²) A | Hydraulic Radius (m) R | R ^{2/3} | AR ^{2/3} | $\left(\frac{n}{AR^{2/3}}\right)^2 \times 10^{-2}$ | n = 0.012 | n = 0.015 | n = 0.019 | n = 0.021 | n = 0.024 |
|------------------|--------------------------------|------------------------------|------------------|-------------------|--|-----------|-----------|-----------|-----------|-----------|
| 150 | 0.02 | 0.038 | 0.112 | 0.002 | 3677 | 5746 | 9219 | 11262 | 14710 | 14710 |
| 200 | 0.03 | 0.050 | 0.136 | 0.004 | 793 | 1239 | 1988 | 2428 | 3172 | 3172 |
| 250 | 0.05 | 0.063 | 0.157 | 0.008 | 214 | 377 | 605 | 739 | 965 | 965 |
| 300 | 0.07 | 0.075 | 0.178 | 0.013 | 912 | 143 | 229 | 279 | 365 | 365 |
| 400 | 0.13 | 0.100 | 0.215 | 0.027 | 19.7 | 30.7 | 49.3 | 60.2 | 78.7 | 78.7 |
| 500 | 0.20 | 0.125 | 0.250 | 0.049 | 5.98 | 9.35 | 15.00 | 18.32 | 23.93 | 23.93 |
| 600 | 0.28 | 0.150 | 0.282 | 0.080 | 2.262 | 3.535 | 5.672 | 6.929 | 9.049 | 9.049 |
| 700 | 0.38 | 0.175 | 0.313 | 0.120 | 0.994 | 1.554 | 2.493 | 3.045 | 3.977 | 3.977 |
| 800 | 0.50 | 0.200 | 0.342 | 0.172 | 0.488 | 0.762 | 1.223 | 1.494 | 1.951 | 1.951 |
| 900 | 0.64 | 0.225 | 0.370 | 0.235 | 0.260 | 0.407 | 0.652 | 0.797 | 1.041 | 1.041 |
| 1000 | 0.79 | 0.250 | 0.397 | 0.312 | 0.148 | 0.232 | 0.372 | 0.454 | 0.594 | 0.594 |
| 1100 | 0.95 | 0.275 | 0.423 | 0.402 | 0.089 | 0.139 | 0.224 | 0.273 | 0.357 | 0.357 |
| 1200 | 1.13 | 0.300 | 0.448 | 0.507 | 0.056 | 0.088 | 0.141 | 0.172 | 0.224 | 0.224 |
| 1300 | 1.33 | 0.325 | 0.473 | 0.627 | 0.037 | 0.057 | 0.092 | 0.112 | 0.146 | 0.146 |
| 1400 | 1.54 | 0.350 | 0.497 | 0.764 | 0.025 | 0.039 | 0.062 | 0.076 | 0.099 | 0.099 |
| 1500 | 1.77 | 0.375 | 0.520 | 0.918 | 0.017 | 0.027 | 0.043 | 0.052 | 0.068 | 0.068 |
| 1600 | 2.01 | 0.400 | 0.543 | 1.091 | 0.012 | 0.019 | 0.030 | 0.037 | 0.048 | 0.048 |
| 1700 | 2.27 | 0.425 | 0.565 | 1.282 | 0.009 | 0.014 | 0.022 | 0.027 | 0.035 | 0.035 |
| 1800 | 2.54 | 0.450 | 0.587 | 1.494 | 0.006 | 0.010 | 0.016 | 0.020 | 0.026 | 0.026 |
| 1900 | 2.83 | 0.475 | 0.609 | 1.725 | 0.005 | 0.008 | 0.012 | 0.015 | 0.019 | 0.019 |
| 2000 | 3.14 | 0.500 | 0.630 | 1.978 | 0.004 | 0.006 | 0.009 | 0.011 | 0.015 | 0.015 |
| 2200 | 3.80 | 0.550 | 0.671 | 2.550 | 0.002 | 0.003 | 0.006 | 0.007 | 0.009 | 0.009 |
| 2400 | 4.52 | 0.600 | 0.711 | 3.217 | 0.0014 | 0.002 | 0.003 | 0.004 | 0.006 | 0.006 |
| 2700 | 5.2 | 0.675 | 0.769 | 4.404 | 0.0007 | 0.0012 | 0.0019 | 0.0023 | 0.0030 | 0.0030 |
| 3000 | 7.07 | 0.750 | 0.825 | 5.832 | 0.0004 | 0.0007 | 0.0011 | 0.0013 | 0.0017 | 0.0017 |
| 3300 | 8.55 | 0.825 | 0.880 | 7.520 | 0.0003 | 0.0004 | 0.0006 | 0.0008 | 0.0010 | 0.0010 |
| 3600 | 10.17 | 0.900 | 0.932 | 9.484 | 0.0002 | 0.0003 | 0.0004 | 0.0005 | 0.0006 | 0.0006 |

To find energy loss in pipe friction for a given Q, multiply Q² by the figure under the proper value of n.

$$\text{Manning Flow Equation: } Q = \left(\frac{AR^{2/3}}{n}\right) \times S^{1/2} \quad \text{Energy Loss} = S = Q^2 \left(\frac{n}{AR^{2/3}}\right)^2$$

Table 5.2 Hydraulic calculation sheet

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 |
|------------|-------------|---------|-----------|--------------|---------------------|---------|----------|------------------------|---------|-----------|-----------------------|----------------------------|--------|---------------------|---------------------|---------------------|---------------------|-----------|----|----|
| Station | Invert m | D mm | H.G. m | Sec- tion | A m ² | K | V m/s | Q m ³ /s | y 2g | E.G. m | S _i m/m | Avg. S _i m/m | L m | H _i m | H _b m | H _m m | H _t m | E.G. m | | |
| 0+ 000.000 | 28.200 | 1800 | 30.000 | 0 | 2.54 | 0.01130 | 2.8 | 7.0 | 0.39 | 30.390 | 0.0127 | 0.0127 | 33.5 | 0.424 | | | | 30.390 | | |
| 0+ 033.528 | 28.624 | 1800 | 30.424 | | 2.54 | 0.01130 | 2.8 | 7.0 | 0.39 | 30.814 | 0.0127 | 0.0127 | 4.7 | 0.059 | 0.056 | | | 30.814 | | |
| 0+ 038.222 | 28.740 | 1800 | 30.540 | | 2.54 | 0.01130 | 2.8 | 7.0 | 0.39 | 30.930 | 0.0127 | 0.0127 | 37.4 | 0.473 | | | | 30.930 | | |
| 0+ 075.590 | 29.212 | 1800 | 31.012 | | 2.54 | 0.01130 | 2.8 | 7.0 | 0.39 | 31.402 | 0.0127 | 0.0266 | 2.3 | 0.061 | | | 0.132 | 31.402 | | |
| 0+ 077.876 | 29.805 | 1400 | 31.205 | | 1.54 | 0.00950 | 4.5 | 7.0 | 1.05 | 32.255 | 0.0406 | 0.0406 | 30.5 | 1.238 | | | | 32.255 | | |
| 0+ 108.356 | 31.043 | 1400 | 32.443 | | 1.54 | 0.00950 | 4.5 | 7.0 | 1.05 | 33.493 | 0.0406 | 0.0406 | 30.5 | 1.238 | | | 0.053 | 33.493 | | |
| 0+ 138.836 | 32.334 | 1400 | 33.734 | | 1.54 | 0.00950 | 4.5 | 7.0 | 1.05 | 34.784 | 0.0406 | 0.0299 | 3.1 | 0.092 | | | 1.085 | 34.784 | | |
| 0+ 141.900 | 33.711 | 1200 | 34.911 | | 1.13 | 0.00785 | 3.1 | 3.5 | 0.49 | 35.401 | 0.0191 | 0.0191 | 35.0 | 0.669 | | | | 35.401 | | |
| 0+ 176.900 | 34.380 | 1200 | 35.580 | | 1.13 | 0.00785 | 3.1 | 3.5 | 0.49 | 36.070 | 0.0191 | 0.0351 | 3.5 | 0.123 | | | | 36.070 | | |
| 0+ 180.421 | 36.402 | 600 | 37.002 | | 0.28 | 0.00636 | 3.5 | 1.0 | 0.64 | 37.492 | 0.0511 | 0.0511 | 35.5 | 1.814 | | | 0.032 | 37.492 | | |
| 0+ 215.892 | 38.248 | 600 | 38.848 | 0 | 0.28 | 0.00636 | 3.5 | 1.0 | 0.64 | 39.338 | 0.0511 | | | | | | | 39.338 | | |

$$n = \text{Variable}$$

$$K = 2\eta(n^2)$$

$$S_i = K \left(\frac{V^2}{2g} \right) R^{4/3}$$

$$\sum H_{\text{friction}} = 6.191$$

$$\sum H_{\text{form}} = 2.657$$

- i) Determine conduit invert (2). In the example we are designing for full flow conditions; therefore, H. G. (4) is at crown of pipe and invert (2) is set by subtracting, D (3) from H. G. (4).
- j) Continue to follow the above procedure taking into account all form head losses.
- k) Complete profile drawing; showing line, grade and pipe sizes. This saves time and usually helps in spotting any design errors.

Energy Losses

Station 0 + 033.528 to 0 + 038.222 (Bend)

$$H_b = K_b \left[\frac{V^2}{2g} \right], \text{ where } K_b = 0.25 \sqrt{\frac{\Phi}{90}}$$

Φ , central angle of bend = 30°

$$K_b = 0.25 \sqrt{\frac{30}{90}} = 0.1443$$

$$\therefore H_b = 0.1433 (0.39) = 0.056 \text{ m}$$

Station 0 + 075.590 to 0 + 077.876 (Transition)

$$H_t = 0.2 \left[\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right], \text{ Expansion}$$

$$= 0.2 (1.05 - 0.39)$$

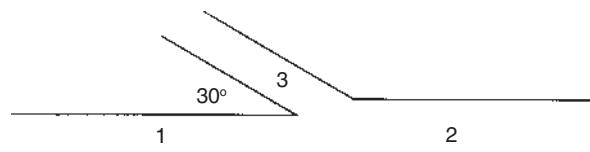
$$= 0.132 \text{ m}$$

Station 0 + 108.356 (man hole)

$$H_m = 0.05 \left[\frac{V^2}{2g} \right]$$

$$= 0.05 (1.05) = 0.053 \text{ m}$$

Station 0 + 138.836 to 0 + 141.900 (Junction)



$$Q_1 = 3.5 \text{ cms}$$

$$A_1 = 1.13 \text{ m}^2$$

$$D_1 = 1200 \text{ mm}$$

$$Q_2 = 7.0 \text{ cms}$$

$$A_2 = 1.54 \text{ m}^2$$

$$D_2 = 1400 \text{ mm}$$

$$Q_3 = 3.5 \text{ cms}$$

$$A_3 = 1.13 \text{ m}^2$$

$$D_3 = 1200 \text{ mm}$$

$$\Phi = 30^\circ$$

- P = • M (Pressure plus momentum laws)

$$\left(H_j + D_1 - D_2 \right) \left(\frac{A_1 + A_2}{2} \right) = \frac{Q_2^2}{A_2 g} - \frac{Q_1^2}{A_1 g} - \frac{Q_3^2 \cos \Phi}{A_3 g}$$

$$(H_j + 1.2 - 1.4) \left(\frac{1.13 + 1.54}{2} \right) = \frac{(7.0)^2}{(1.54)(9.81)} - \left(\frac{3.5^2}{1.13(9.81)} \right) - \left(\frac{3.5^2 \cos 30^\circ}{1.13(9.81)} \right)$$

$$1.335 H_j - 0.2 (1.335) = 3.243 - 1.105 - 0.957$$

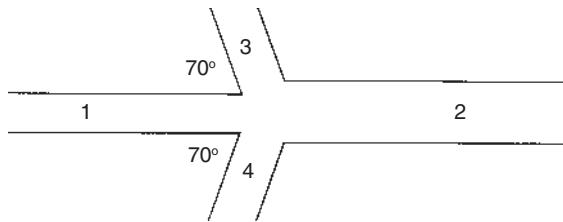
$$1.335 H_j - 0.267 = 1.181$$

$$H_j = 1.085 \text{ m}$$



Fittings and elbows are easily fabricated in all sizes.

Station 0 + 176.900 to 0 + 180.421 (Junction)



$$\begin{array}{lll}
 Q_1 = 1.0 \text{ m}^3/\text{s} & Q_2 = 3.5 & Q_3 = 1.5 \\
 A_1 = 0.28 \text{ m}^2 & A_2 = 1.13 & A_3 = 0.64 \\
 D_1 = 600 \text{ mm} & D_2 = 1200 & D_3 = 900 \\
 & & \theta_3 = 70^\circ \\
 & & \theta_4 = 70^\circ
 \end{array}
 \quad
 \begin{array}{lll}
 Q_4 = 1.0 & & \\
 A_4 = 0.28 & & \\
 D_4 = 600 & & \\
 & & \theta_4 = 70^\circ
 \end{array}$$

$$\begin{aligned}
 (H_j + D_1 - D_2) \left(\frac{A_1 + A_2}{2} \right) &= \frac{Q_2^2}{A_2 g} \\
 &\quad - \frac{Q_1^2}{A_1 g} - \frac{Q_3^2 \cos \theta_3}{A_3 g} - \frac{Q_4^2 \cos \theta_4}{A_4 g}
 \end{aligned}$$

$$\begin{aligned}
 (H_j + 0.6 - 1.2) \left(\frac{0.28 + 1.13}{2} \right) &= \frac{(3.5)^2}{(1.13)(9.81)} - \frac{(1.0)^2}{(0.28)(9.81)} \\
 &\quad - \frac{(1.5)^2 \cos 70^\circ}{(0.64)(9.81)} - \frac{(1.0)^2 \cos 70^\circ}{(0.28)(9.81)}
 \end{aligned}$$

$$0.705 H_j - 0.6 (0.705) = 1.105 - 0.364 - 0.123 - 0.125$$

$$0.705 H_j - 0.423 = 0.493$$

$$H_j = 1.299 \text{ m}$$

Station 0 + 215.892 (manhole)

$$H_m = .05 \left(\frac{V^2}{2g} \right) = 0.5 (0.64)$$

$$= 0.032 \text{ m}$$

Total friction H_f throughout the system = 6.191 m

Total form losses = 2.657 m

In this example, the head losses at junctions and transition could also have been accommodated by either increasing the pipe diameter or increasing the slope of the pipe.

This backwater example was designed under full flow conditions but could also have been designed under pressure; allowing surcharging in the manholes, which would have reduced the pipe sizes. Storm sewer systems, in many cases, can be designed under pressure to surcharge to a tolerable level.

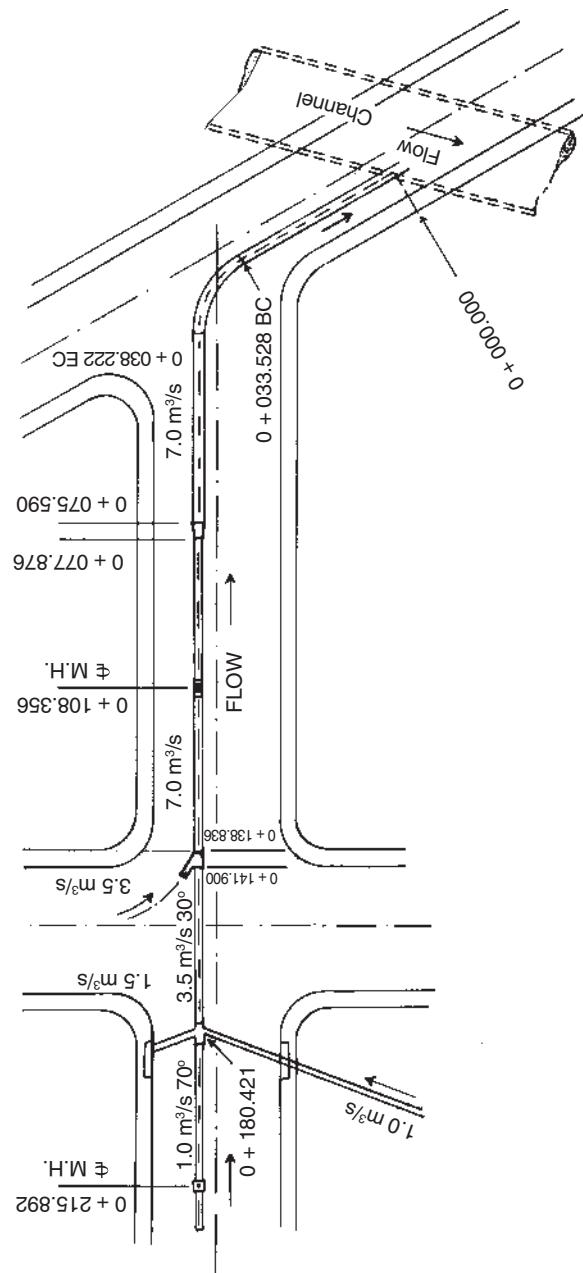


Figure 5.1 Plan and profile for storm sewer

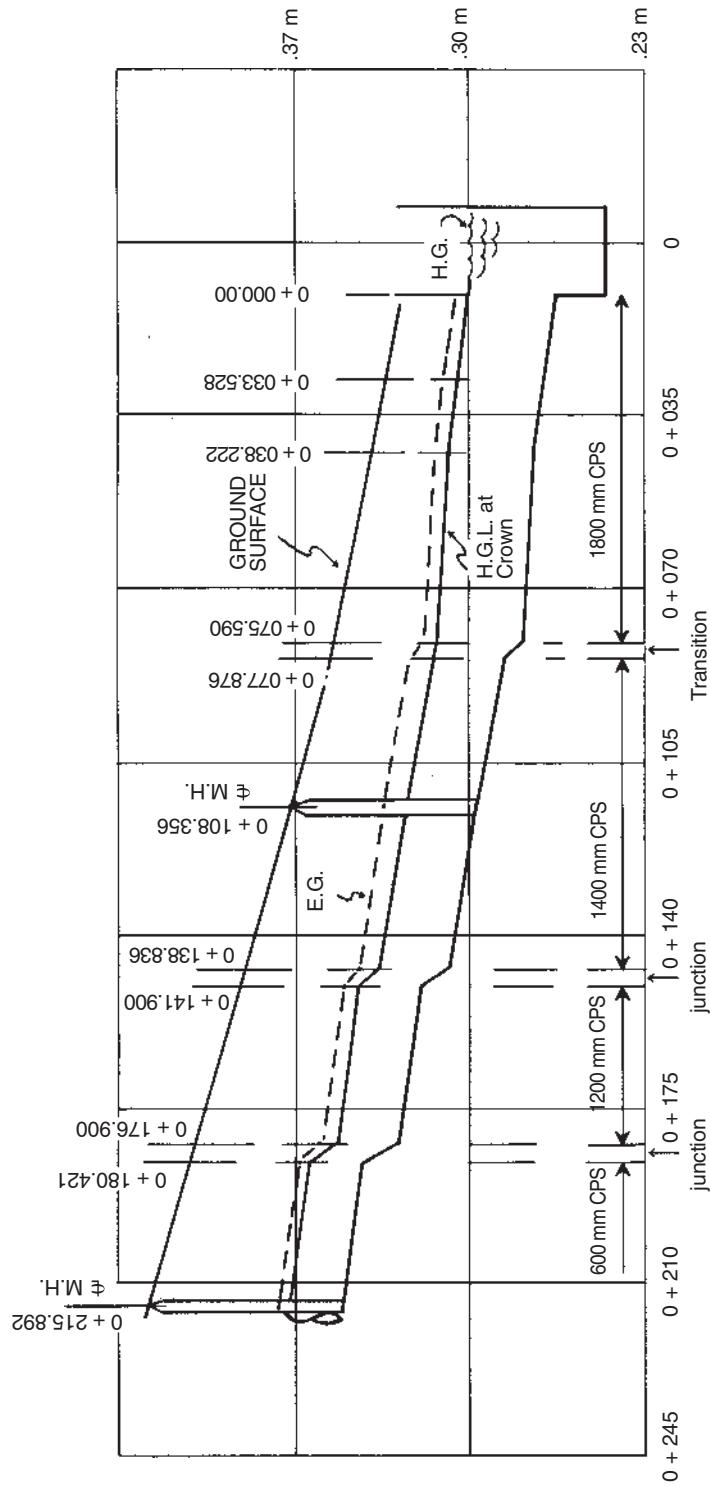


Figure 5.2 Plan and profile for storm sewer

METHODS OF DETERMINING EQUIVALENT HYDRAULIC ALTERNATIVES

A method has been developed to aid the designer in quickly determining equivalent pipe sizes for alternative material, rather than computing the backwater profiles for each material.

The derivation shown below allows the designer to assign representative values for loss coefficients in the junctions and length of average reach between the junctions, and develop a relationship for pipes of different roughness coefficients. In this manner the designer need only perform a detailed hydraulic analysis for one material, and then relatively quickly determine conduit sizes required for alternative materials. The relationships for hydraulic equivalent alternatives in storm sewer design may be derived from the friction loss equation.

The total head loss in a sewer system is comprised of junction losses and friction losses:

$$H_T = H_j + H_f$$

$$\begin{aligned} \text{where: } H_j &= K_j \frac{V^2}{2g} \\ &= K_j \frac{Q^2}{A^2 2g} \\ &= K_j \frac{Q^2 16}{\pi^2 D^4 2g} \end{aligned}$$

where:

$$H_f = \frac{2n^2 L V^2}{R^{4/3} 2g} = \frac{13 n^2 L Q^2 (16)}{2g \pi^2 D^{16/3}} \text{ for } K_f = 2n^2$$

$$\begin{aligned} H_T &= H_j + H_f \\ &= \frac{16Q^2 K_j}{2g \pi^2 D^4} + \frac{13 n^2 L Q^2 (16)}{2g \pi^2 D^{16/3}} \\ &= \frac{8Q^2}{g \pi^2} \left[\frac{K_j D^{4/3} + 13 n^2 L}{D^{16/3}} \right] \end{aligned}$$

Thus, for comparison of concrete and steel:

$$\frac{8Q^2}{g\neq} \left[\frac{K_j(D_c)^{4/3} + 13(n_c)^2 L}{(D_c)^{16/3}} \right] = \frac{8Q^2}{g\neq} \left[\frac{K_j(D_s)^{4/3} + 13(n_s)^2 L}{(D_s)^{16/3}} \right]$$

The flow Q for each conduit will be the same, therefore the relationship simplifies to:

$$\frac{K_j(D_c)^{4/3} + 13(n_c)^2 L}{(D_c)^{16/3}} = \frac{K_j(D_s)^{4/3} + 13(n_s)^2 L}{(D_s)^{16/3}}$$

Average values for conduit length between manholes (L), and junction loss coefficient (K_j), must next be selected. Representative values may be derived for the hydraulic calculations that will have already been performed for one of the materials.

In this example the average conduit length is 90 metres with an average junction loss coefficient of 1.0. With the selected L, n and K_j values the equations are determined for a series of pipe diameters. The results are shown in Table 5.3. These figures are then plotted on semi-log paper, from which hydraulically equivalent materials may be easily be selected. (Figures 5.3 and 5.4)



Combination increaser, manhole and elbow in one length of pipe.

Table 5.3 Methods of determining equivalent alternatives

| Diameter (mm) | Junction and Friction Losses | | | n values |
|------------------|------------------------------------|------------------------------------|--|----------|
| | $K_j = 1.0$ | | $L = 90m$ | |
| | Smooth Pipe $n = 0.012$ | Annular CSP Pipe $n = 0.024$ | Helical CSP Pipe n var. (see Table 4.9) | |
| D | $\frac{D^{4/3} + 0.168}{D^{16/3}}$ | $\frac{D^{4/3} + 0.674}{D^{16/3}}$ | $\frac{D^{4/3} + 1170 n^2}{D^{16/3}}$ | |
| 200 | 1525.30 | 4226.21 | 1525.30 | 0.012 |
| 250 | 529.86 | 1351.46 | 486.12 | 0.011 |
| 300 | 227.03 | 537.74 | 245.01 | 0.013 |
| 400 | 61.39 | 128.38 | 95.04 | 0.019 |
| 500 | 22.79 | 43.17 | 26.61 | 0.015 |
| 600 | 10.28 | 17.99 | 13.50 | 0.018 |
| 700 | 5.29 | 8.68 | 6.71 | 0.018 |
| 800 | 3.00 | 4.66 | 3.98 | 0.02 |
| 900 | 1.82 | 2.71 | 2.43 | 0.021 |
| 1000 | 1.17 | 1.67 | 1.52 | 0.021 |
| 1200 | 0.55 | 0.74 | 0.68 | 0.021 |
| 1400 | 0.29 | 0.37 | 0.35 | 0.021 |
| 1600 | 0.17 | 0.21 | 0.19 | 0.021 |
| 1800 | 0.10 | 0.12 | 0.12 | 0.021 |
| 2000 | 0.07 | 0.08 | 0.08 | 0.021 |
| 2200 | 0.05 | 0.05 | 0.05 | 0.021 |
| 2400 | 0.03 | 0.04 | 0.03 | 0.021 |

Note: Pipe diameter in metres in above equations.



Philadelphia Airport, 16 x 26mm fiber-bonded, full bituminous coated and full paved CSP with semi-corrugated bands with O-ring gaskets, provides storm drainage for airport— 5800 m of 2000mm, 2200mm, 2400mm, 2700mm diameters, 14 - 16 gauge, 2 - 3m of cover.

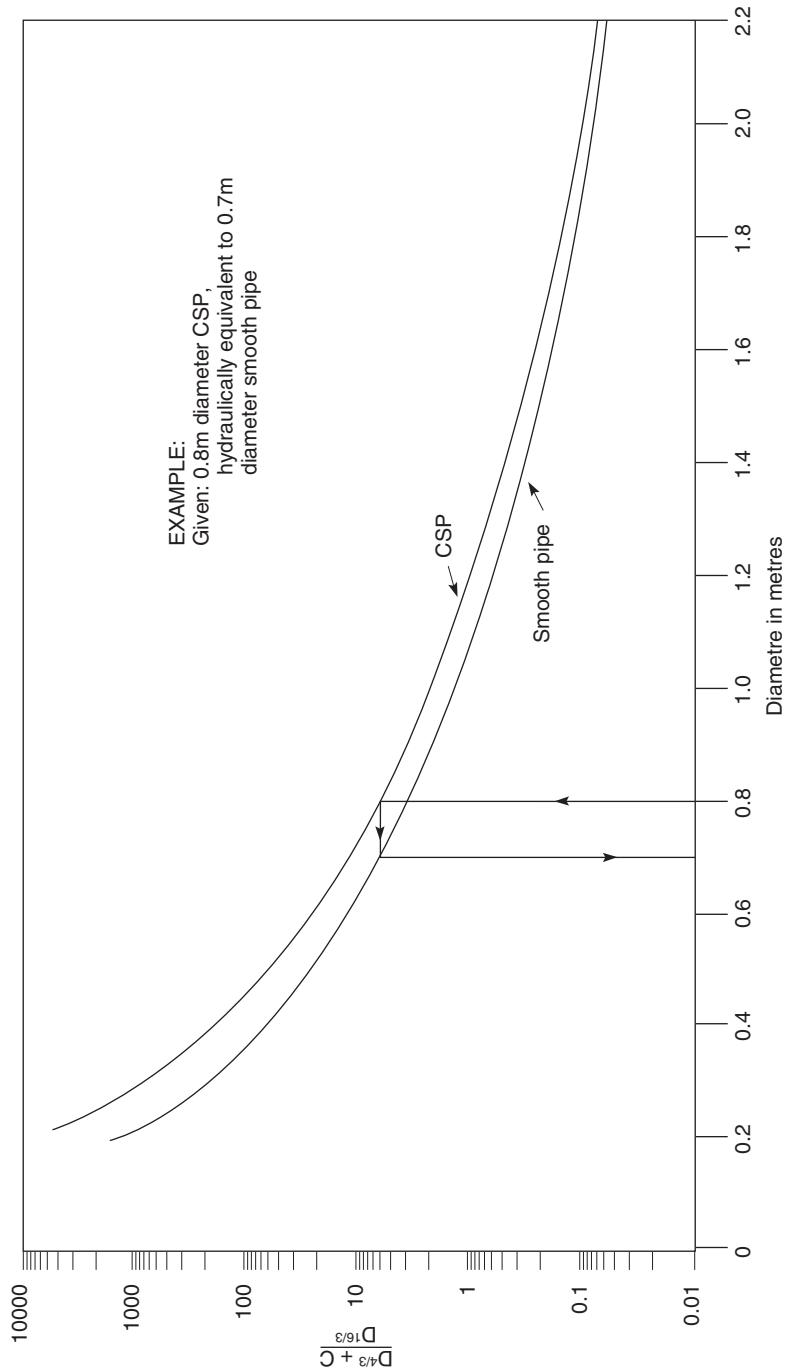


Figure 5.3 Equivalent alternatives with annular CSP where $C = 13\text{m}^2\text{L}$

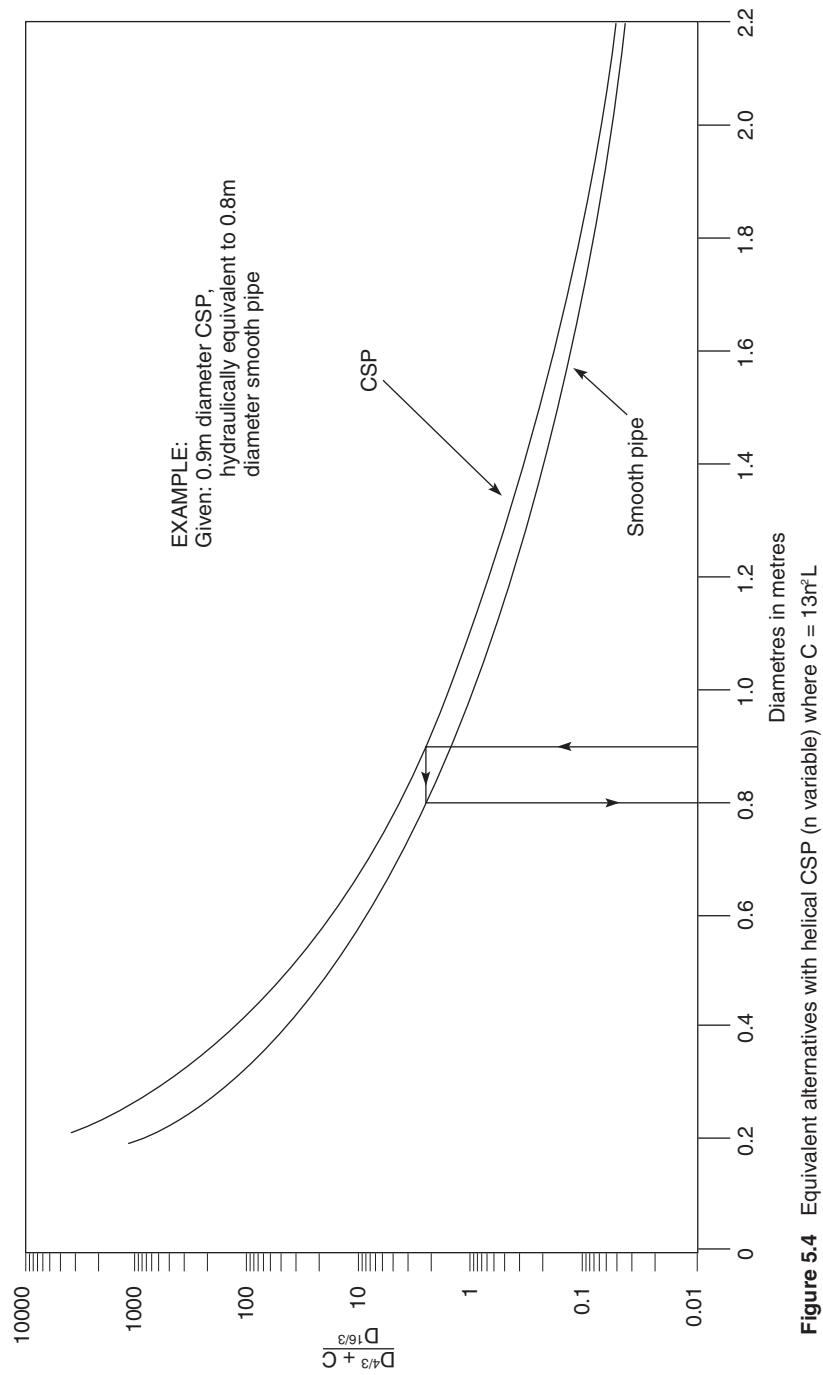


Figure 5.4 Equivalent alternatives with helical CSP (n variable) where $C = 13^n L$

DESIGN OF STORM DRAINAGE FACILITIES

System Layout

The storm drainage system layout should be made in accordance with the urban drainage objectives, following the natural topography as closely as possible. Existing natural drainage paths and watercourses such as streams and creeks should be incorporated into the storm drainage system. Thus the storm design should be undertaken prior to finalization of the street layout in order to effectively incorporate the major-minor drainage concepts.

Topographic maps, aerial photographs, and drawings of existing services are required before a thorough storm drainage design may be undertaken.

Existing outfalls within the proposed development and adjacent lands for both the minor and major system should be located. Allowances should be made for external lands draining through the proposed development both for present conditions and future developments.

The design flows used in sizing the facilities that will comprise the drainage network are based on a number of assumptions. Flows that will occur under actual conditions will thus be different from those estimated at the design stage; "the designer must not be tempted by the inherent limitations of the basic flow data to become sloppy in the hydraulic design." (1) Also the designer should not limit his investigation to system performance under the design storm conditions, but should assure that in cases where sewer capacities are exceeded such incidents will not create excessive damage.

This requirement can only be practically achieved if the designer realizes that a dual drainage system exists, comprised of the minor system and the major system. Utilizing both systems, the pipe system may be provided for smaller, more frequent rainfall events, and an overland system for extreme rainfall events.

In the layout of an effective storm drainage system, the most important factor is to assure that a drainage path both for the minor and major systems be provided to avoid flooding and ponding in undesirable locations.

Minor System

The minor system consists chiefly of the storm sewer comprised of inlets, conduits, manholes and other appurtenances designed to collect and convey into a satisfactory system outfall, storm runoff for frequently occurring storms (2 to 5 year design).

Storm sewers are usually located in rights-of-way such as roadways and easements for ease of access during repair or maintenance operations.

Major System

The major drainage system will come into operation when the minor system's capacity is exceeded or when inlet capacities significantly control discharge to the minor system. Thus, in developments where the major system has been planned, the streets will act as open channels draining the excess storm water. The depth of flow on the streets should be kept within reasonable limits for reasons of safety and convenience. Consideration should be given to the area of flooding and its impact on various street classifications and to public and private property.

Typical design considerations are given in Table 5.4.

Table 5.4 Typical maximum flow depths

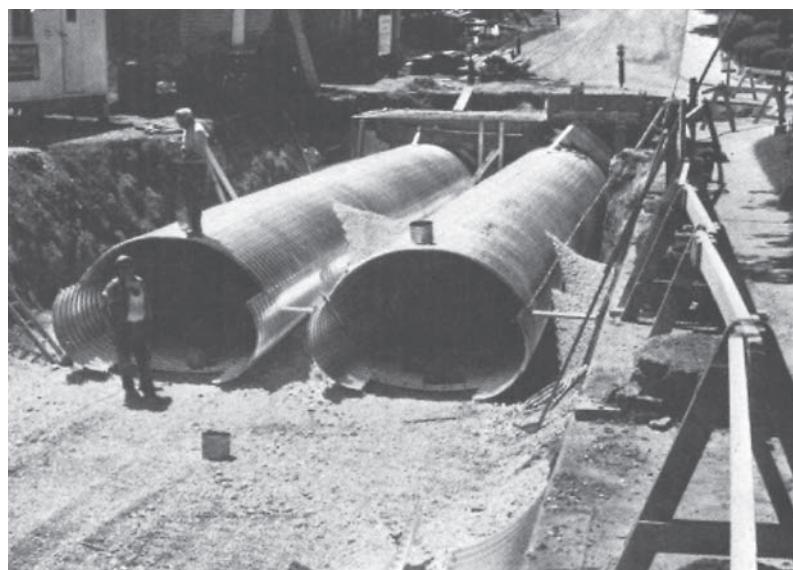
| Location* | Storm Return Frequency (Years) | | |
|-----------------------------------|--|--|--|
| | 5 | 25 | 40 |
| Walkways, Open spaces | Minor surface flow up to 25mm deep on walkways | As required for overland flow outlets | As required for overland flow outlets |
| Minor, Local and Feeder Roads | 1m wide in gutters or 100mm deep at low point catch basins | 100mm above crown | 200mm above crown |
| Collector and Industrial Roads | Minor surface flow (25mm) | up to crown | 100mm above crown |
| Arterial Roads | Minor Surface flow (25mm) | 1 lane clear | up to crown |

*In addition to the above, residential buildings, public, commercial and industrial buildings should not be inundated at the ground line for the 100 year storm, unless buildings are flood-proofed.

To prevent the flooding of basement garages, driveways will have to meet or exceed the elevations corresponding to the maximum flow depth at the street.

The flow capacity of the streets may be calculated from the Manning equation, or Figure 5.5 may be used to estimate street flows.

When designing the major system it should be done in consideration of the minor system, with the sum of their capacities being the total system's capacity. The minor system should be first designed to handle a selected high frequency storm, (i.e., 2-year) next the major system is designated for a low frequency of flood storm, (i.e., 100-year). If the roadway cannot handle the excess flow, the minor system should be enlarged accordingly.



Multiple inline storage installation.

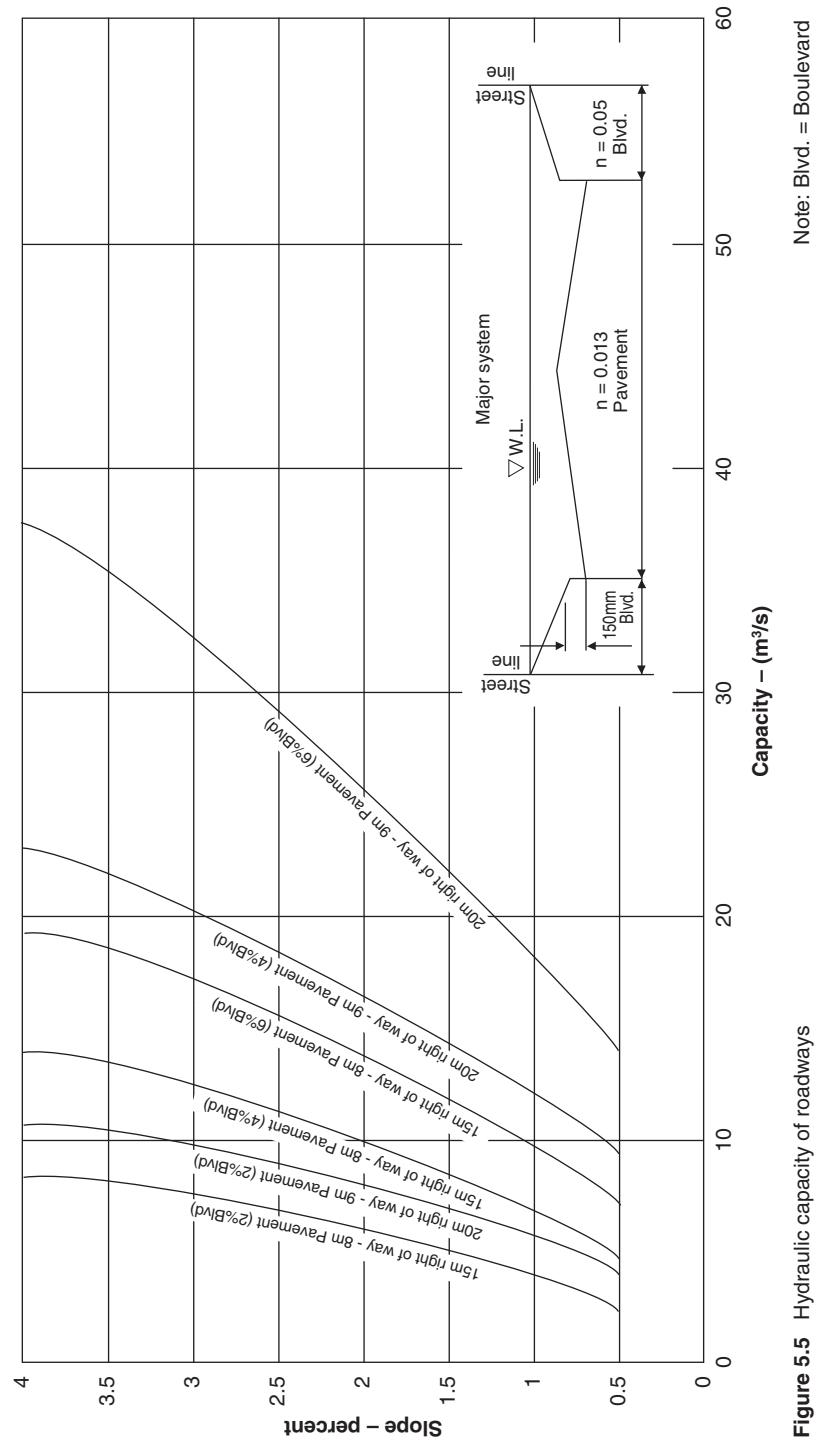


Figure 5.5 Hydraulic capacity of roadways

HYDRAULIC DESIGN EXAMPLE OF MINOR-MAJOR SYSTEM

Description of Site

The site for this design example is shown on Figure 5.6.

The site is about 15 hectares in size consisting of single family and semi detached housing as well as a site for a public school. The site slopes generally from west to east, where it is bounded by a major open water course. To accommodate the principles of the “minor-major” storm drainage systems, the streets have been planned to conform as much as possible to the natural contours of the lands. Where sags in roadways between intersections could not be avoided, overflow easements or walkways have been provided to permit unobstructed surface runoff during major storms, as shown on Figure 5.7.

Selected Design Criteria

Minor System

Based on a reasonable level of convenience to the public, a two-year design curve is considered adequate as a design basis for the minor system within this development.



Storm Sewer installation involved 1300 m of full bituminous coated full paved pipe arch.

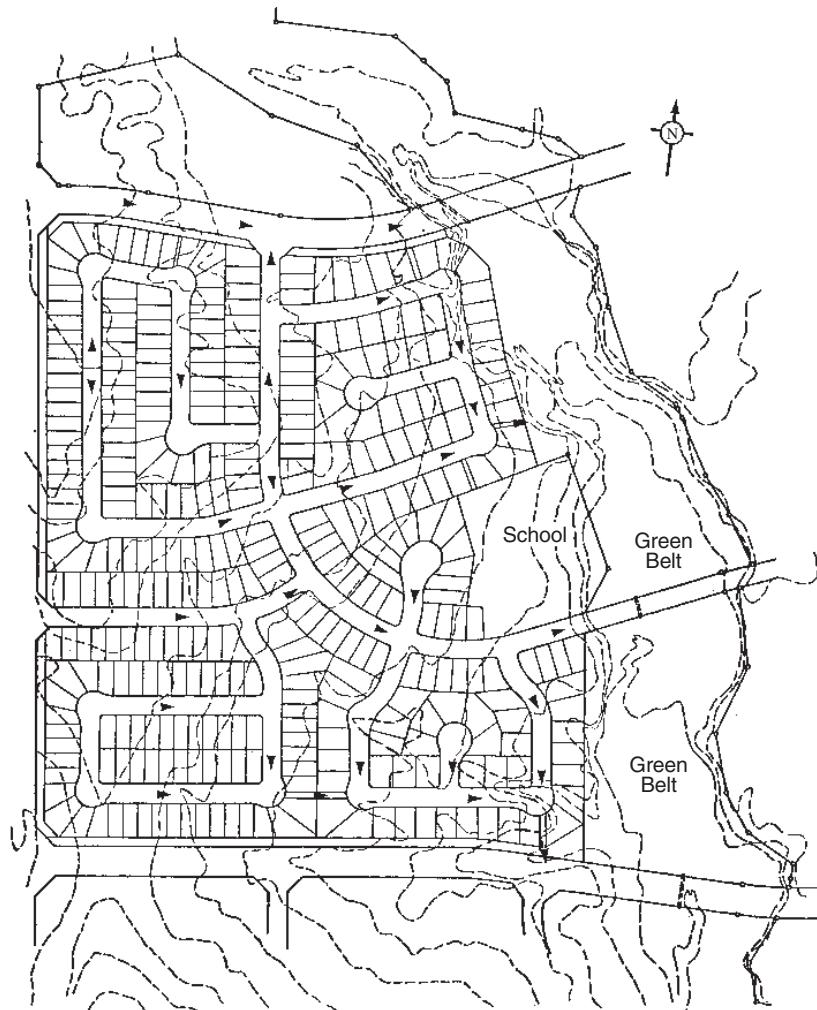


Figure 5.6 Site plan with route of surface runoff

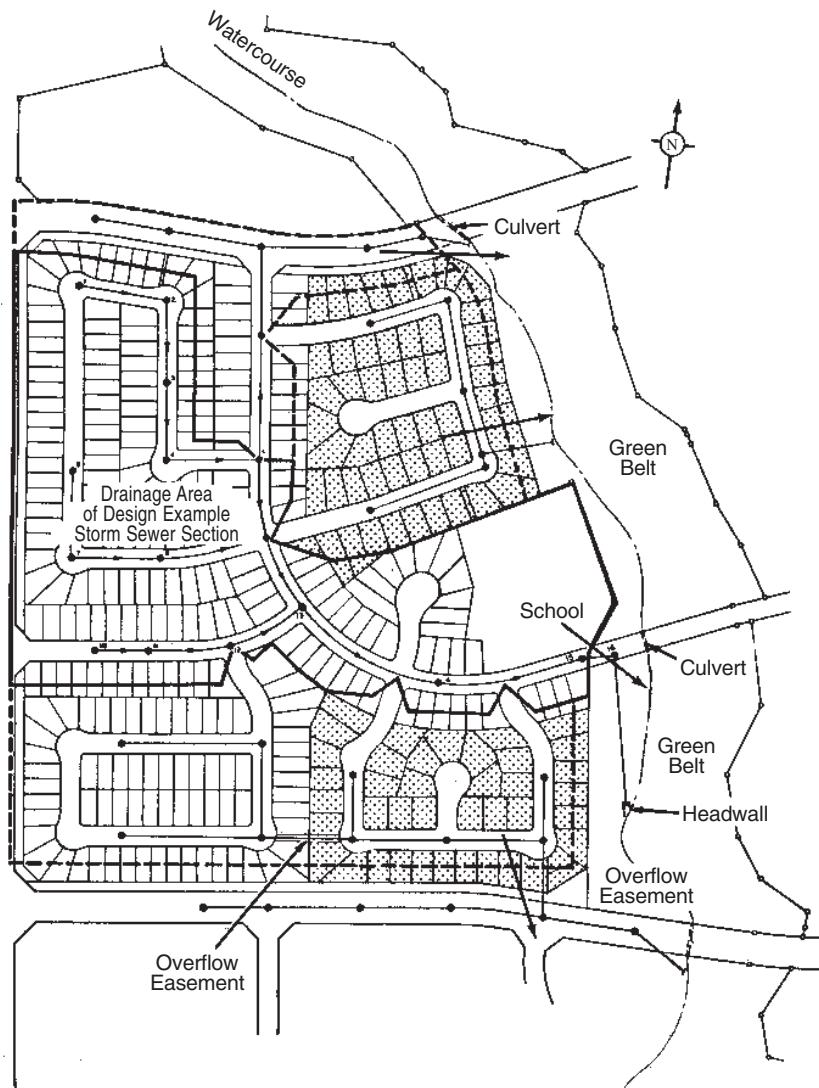


Figure 5.7 Storm drainage areas

Major System

The major (or overflow) system will be checked together with the minor system against a 100-year storm intensity. The combination of these two systems shall be able to accommodate a 100-year storm runoff.

Minor System

For the limited extent of area involved, designing on the principles of the minor-major drainage concept without gravity connections to foundation drains permits considerable tolerance in the degree of accuracy of runoff calculations such that the rational formula $Q = k \cdot C \cdot i \cdot A$ is considered ad-

equate. The values for the two year rainfall intensity curve obtained from local records are shown in Table 5.5.

Table 5.5 Rainfall intensity duration frequency

| Time (Min) | 2 Year Return (mm/hr) | 100 Year Return (mm/hr) |
|---------------|--------------------------|----------------------------|
| 5 | 105 | 262 |
| 10 | 72 | 179 |
| 15 | 57 | 146 |
| 20 | 48 | 122 |
| 25 | 42 | 109 |
| 30 | 37 | 97 |
| 35 | 34 | 89 |
| 40 | 30 | 81 |
| 45 | 28 | 74 |
| 50 | 26 | 69 |
| 55 | 24 | 64 |
| 60 | 22 | 59 |
| 65 | 21 | 55 |
| 70 | 19 | 51 |
| 75 | 18 | 47 |
| 80 | 16 | 44 |
| 85 | 15 | 41 |
| 90 | 13 | 39 |
| 95 | 12 | 38 |
| 100 | 11 | 33 |
| 125 | 10 | 32 |
| 150 | 9 | 25 |
| 175 | 7 | 23 |
| 200 | 7 | 22 |

The following steps should be followed in the hydraulic design of the minor system:

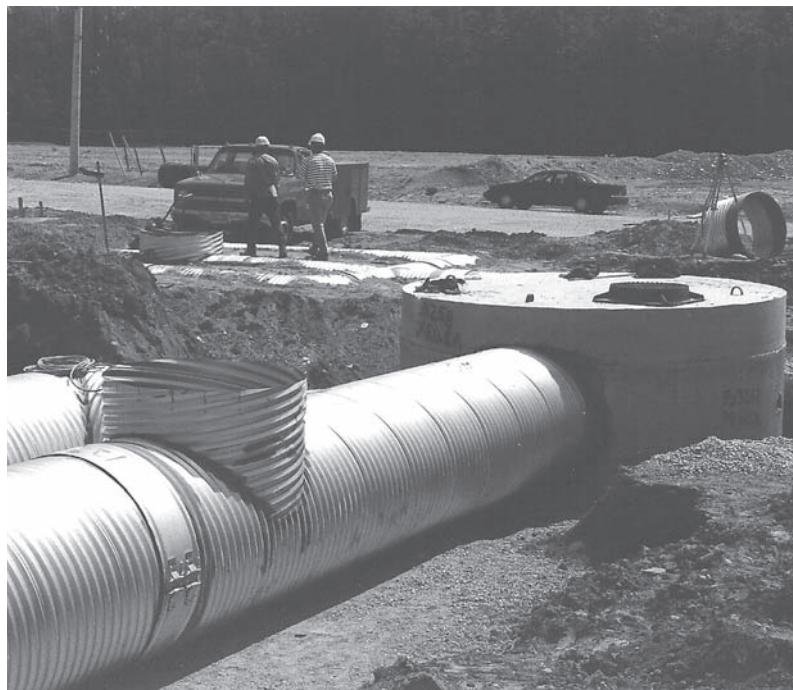
1. A drainage area map should be prepared indicating the drainage limits for the site, external tributary areas, location of imported minor system and carryover flows, proposed minor-major system layout and direction of surface flow.
2. The drainage area should be divided into sub-areas tributary to the proposed storm sewer inlets. In this case the inlet shall be located at the upstream end of each pipe segment.
3. The coverage of each sub-area should be calculated.
4. The appropriate runoff coefficient should be developed for each sub-area. The example has been simplified in that impervious areas discharging to grass areas have been given a runoff coefficient equal to the grassed area runoff coefficient. The runoff coefficient in this example has been determined based on 0.20 for grassed areas and areas discharging to grass such as roof, patios and sidewalks) and 0.95 for impervious surfaces (streets and driveways), which for this site results in an average runoff coefficient of 0.35 for all the sub-areas.
5. The required capacity of each inlet should be calculated using the rational method, with the initial time of concentration and the corresponding intensity. In this example,
 $T_c = 10$ minutes.
 $i = 72 \text{ mm/hr}$ (2-year storm) (Table 5.5).
 Inlets will be located at the upstream manhole for each length of conduit.

6. Commencing at the upstream end of the system, the discharge to be carried by each successive segment in a downstream direction is calculated. The initial time of concentration is 10 minutes at the most upstream inlet. Added to this value is the required travel time in the conduit to the next inlet. The resulting time of concentration is then used to determine a new intensity at that point.

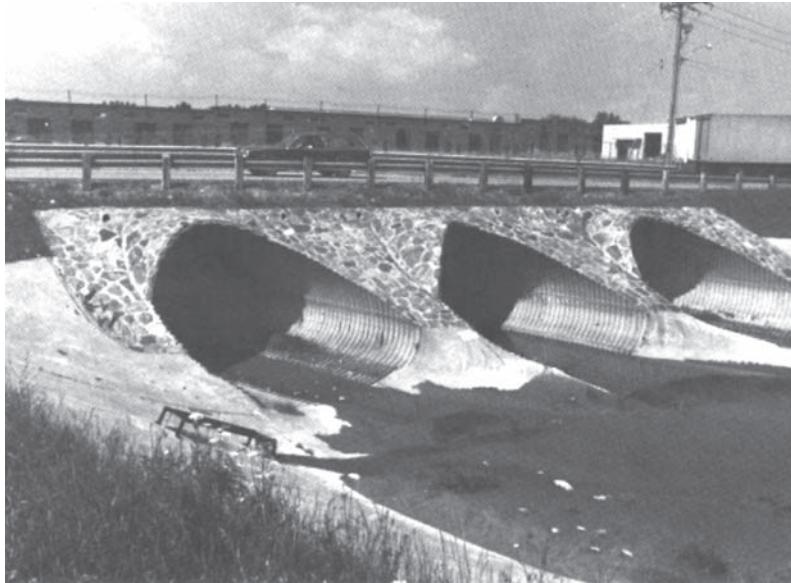
Also, a weighted area x C value must be determined at each successive inlet.

At a confluence of two or more conduits, the longest time of concentration is selected and the procedure continues downstream. The above computations are summarized in Table 5.6.

7. With computed discharges at the upstream end of each pipe segment, a tentative pipe size to accommodate friction losses only is selected using the friction flow charts in Chapter 4. In this design example, a helical 68mm x 13mm CSP with variable roughness coefficient (Table 4.9) has been selected as the conduit material. The corresponding velocities for the expected flow are determined to calculate the pipe flow time. This time added to the upstream time of concentration results in the new time of concentration for the downstream segment as described in Step 6. Design velocities in storm sewers should be a minimum of 1.0 m/s when flowing half full to full to attain self cleaning velocities and to prevent deposition, to a maximum of 4.5 m/s to avoid erosive damage to the conduit.



Recharge trench installation showing junction box.



Culvert design technology and open-channel flow design are increasingly applied to urban storm water management. Triple structural plate pipe-arches enclose stream under roadway, and industrial land development.

Note: If upon completion of the hydraulic design (and backwater calculations), the times of concentrations have varied enough to alter the discharges, new flow values should be determined. In most cases the slight variance in the T_c will not significantly affect the peak flows.

8. As the preliminary design proceeds downstream, some account must be made for the manhole and junction losses. Certain rules of thumb may be used before the detailed hydraulic analysis. In this design example the following manhole drops were assumed:

15 mm for straight runs

45 mm for 45° junctions

75 mm for 45° to 90° junctions

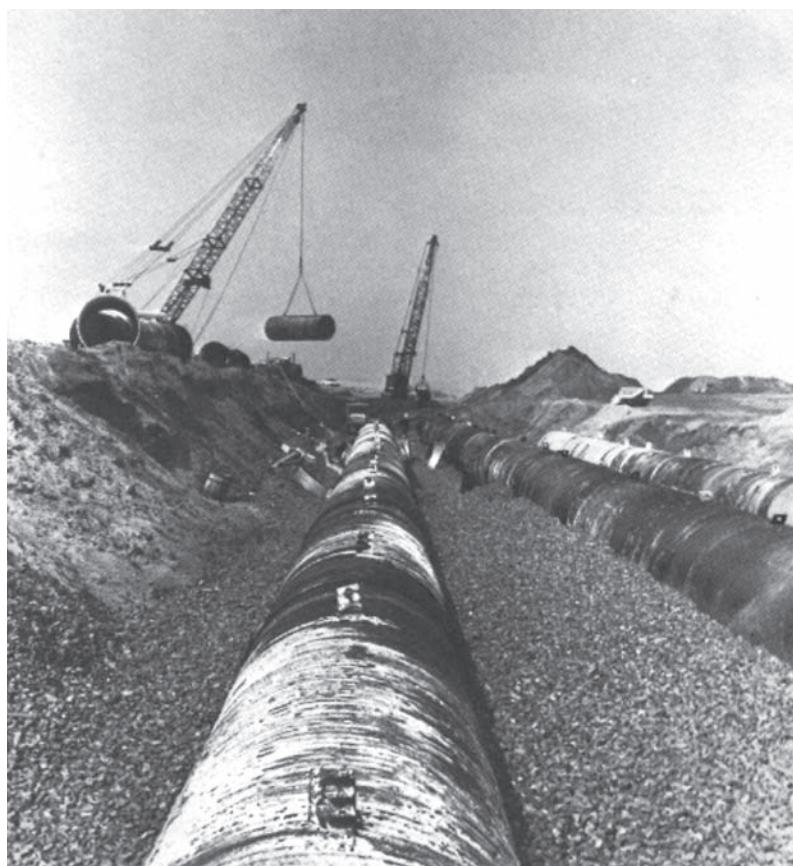
Also crowns of incoming and outgoing pipes at manholes were kept equal where the increase in downstream diameter met or exceeded the above manhole drops.

The preliminary minor system design is shown in Table 5.6 with the tentative pipe sizes and manhole drops.

9. The hydraulic analysis should next be performed on the proposed minor system to ensure that it operates as expected. The hydraulic grade is set at the crown of the outlet conduit, with hydraulic calculations proceeding upstream. The energy loss equations shall be used following the same procedure as in the Hydraulic section. The detailed hydraulic calculations are computed for each station, on pages 153 and 154, with the results summarized in Table 5.7. In this example the initial pipe sizes did not change, but rather manhole drops were adjusted to account for the junction losses. If junction losses would have resulted in the elevation of the pipe crown exceeding the minimum cover criterion, then the

hydraulic grade line may have been lowered by increasing the pipe size. The hydraulic grade line may be permitted to exceed the crown where some surcharging in the storm system can be tolerated.

10. The designer may now estimate the required pipe sizes for a minor system for an alternative conduit material or roughness coefficient. There is no need to perform a detailed hydraulic analysis for the alternative conduit, but rather use the method of "Equivalent Alternatives" as described earlier in this chapter. In this example the average length of conduit is estimated to be 90m with an average manhole junction loss coefficient of 1.0. The alternative conduit will have constant $n = .012$. Therefore the alternative material may be determined. The results are summarized in Table 5.8.



Large storm drain projects under runways at a major airport.



Increasers are easily fabricated for correct field location.



Pipe-arch sewer installation in a residential area satisfying minimum headroom requirement; however it has adequate capacity and strength.

Table 5.6 Preliminary storm sewer design

| M.H. From To | M.H. Area (ha) | Runoff C | Total Section | | | Intensity I | Length of Pipe (m) | Size Pipe (mm) | Slope % | Fall (m) | M.H. Drop (mm) | Up Stream (m) | Down Stream (m) | Actual Cap. Q(m³/s) | Inverts | Time (Entry: 10 Min.) | | | |
|--------------------|----------------------|-------------|------------------|-------|-----------------------------|----------------|-----------------------------|----------------------|------------|-------------|----------------------|---------------------|-----------------------|---------------------------|---------|--------------------------|--------|-------|--|
| | | | A × C | A × C | A × C (mm³/hr) (m³/s) | | | | | | | | | | | Sect. | Accum. | Min. | |
| 1 | 2 | 0.74 | 0.35 | 0.26 | 0.26 | 72 | 0.05 | 90 | 200 | 0.84 | 0.76 | 231.590 | 230.830 | 0.05 | 1.03 | 1.47 | 11.47 | | |
| 2 | 3 | 1.10 | 0.35 | 0.39 | 0.65 | 67 | 0.12 | 80 | 300 | 1.30 | 1.04 | 75 | 230.755 | 229.715 | 0.12 | 1.71 | 0.77 | 12.24 | |
| 3 | 4 | 1.04 | 0.35 | 0.36 | 1.01 | 65 | 0.18 | 81 | 400 | 0.98 | 0.79 | 75 | 229.640 | 228.850 | 0.18 | 1.58 | 0.85 | 13.09 | |
| 4 | 5 | 0.83 | 0.35 | 0.29 | 1.30 | 62 | 0.22 | 93 | 400 | 1.50 | 1.30 | 75 | 228.775 | 227.475 | 0.22 | 1.96 | 0.79 | 13.88 | |
| 6 | 7 | 1.06 | 0.35 | 0.37 | 0.37 | 72 | 0.07 | 90 | 200 | 1.70 | 1.53 | 231.610 | 230.080 | 0.07 | 1.47 | 1.04 | 11.04 | | |
| 7 | 8 | — | — | — | — | — | — | — | 200 | 1.70 | 1.53 | 75 | 230.005 | 228.475 | 0.07 | 1.47 | 1.04 | 12.08 | |
| 8 | 9 | 1.50 | 0.35 | 0.53 | 0.90 | 66 | 0.16 | 75 | 300 | 2.20 | 1.65 | 45 | 228.430 | 226.780 | 0.16 | 2.23 | 0.56 | 12.64 | |
| 10 | 11 | 1.80 | 0.35 | 0.63 | 0.63 | 72 | 0.13 | 90 | 300 | 1.40 | 1.26 | 230.360 | 229.100 | 0.13 | 1.77 | 0.86 | 10.86 | | |
| 11 | 12 | 0.71 | 0.35 | 0.25 | 0.88 | 69 | 0.17 | 83 | 300 | 2.40 | 1.99 | 15 | 229.085 | 227.095 | 0.17 | 2.32 | 0.60 | 11.46 | |
| 12 | 13 | 0.42 | 0.35 | 0.15 | 1.03 | 67 | 0.19 | 81 | 400 | 1.10 | 0.89 | 75 | 227.020 | 226.130 | 0.19 | 1.68 | 0.80 | 12.26 | |
| 5 | 9 | 0.43 | 0.35 | 0.15 | 1.45 | 60 | 0.24 | 81 | 500 | 0.76 | 0.62 | 75 | 227.400 | 226.780 | 0.24 | 1.47 | 0.92 | 14.80 | |
| 9 | 13 | 0.53 | 0.35 | 0.19 | 2.54 | 58 | 0.40 | 81 | 600 | 0.62 | 0.50 | 150 | 226.630 | 226.130 | 0.41 | 1.41 | 0.96 | 15.76 | |
| 13 | 14 | 2.28 | 0.35 | 0.80 | 4.37 | 56 | 0.67 | 150 | 600 | 1.70 | 2.55 | 75 | 226.055 | 223.505 | 0.68 | 2.32 | 1.09 | 16.85 | |
| 14 | 15 | 0.55 | 0.35 | 0.19 | 4.56 | 55 | 0.69 | 150 | 600 | 1.80 | 2.70 | 15 | 223.490 | 220.790 | 0.70 | 2.39 | 1.06 | 17.91 | |
| 15 | 16 | 2.35 | 0.20 | 0.47 | 5.03 | 53 | 0.74 | 35 | 700 | 1.20 | 0.42 | 75 | 220.715 | 220.295 | 0.74 | 1.99 | 0.28 | 18.19 | |
| 16 | Outfall | — | — | — | 5.03 | 53 | 0.74 | 150 | 800 | 0.68 | 1.02 | 75 | 220.220 | 219.200 | 0.74 | 1.61 | 1.58 | 19.77 | |

Q = Flow

A = Area in Hectares

C = Coefficient of Runoff

I = Intensity of Rainfall for Period in mm/hr



Installing 1,800mm diameter CSP to be used as an underground detention chamber for stormwater runoff.

Table 5.7 Hydraulic calculation sheet

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 |
|--------|-------------|---------|-----------|--------------------------------|---------------------|---------|----------|------------------------|------------------|-----------|--------------|-------------------|--------|------------|------------|------------|------------|------------|-----------|----|
| M.H. | Invert m | D mm | H.G. m | Sec- tion m ² | A m ² | K | V m/s | Q m ³ /s | $\frac{V^2}{2g}$ | E.G. m | S_i m/m | Avg. S_i m/m | L m | H_i m | H_b m | H_i m | H_m m | H_t m | E.G. m | |
| Outlet | 219.200 | 800 | 220.000 | 0 | 0.50 | 0.00636 | 1.473 | 0.74 | 0.111 | 220.111 | 0.0060 | 0.0060 | 150 | 0.900 | 0.033 | 0.016 | 220.111 | | | |
| 16 | 220.149 | 800 | 220.949 | 0 | 0.50 | 0.00636 | 1.473 | 0.74 | 0.111 | 221.060 | 0.0060 | 0.0060 | 150 | 0.900 | 0.033 | 0.016 | 221.060 | | | |
| 15 | 220.593 | 700 | 221.293 | 0 | 0.38 | 0.00636 | 1.924 | 0.74 | 0.189 | 221.482 | 0.0123 | 0.0092 | 35 | 0.322 | 0.015 | 0.022 | 221.482 | 221.482 | | |
| 14 | 223.478 | 600 | 224.078 | 0 | 0.28 | 0.00636 | 2.442 | 0.69 | 0.304 | 224.382 | 0.0243 | 0.0183 | 150 | 2.745 | 0.025 | 0.015 | 224.382 | 224.382 | | |
| 13 | 227.428 | 600 | 228.028 | 0 | 0.28 | 0.00636 | 2.371 | 0.67 | 0.287 | 228.315 | 0.0229 | 0.0236 | 150 | 3.540 | 0.034 | 0.376 | 228.315 | | | |
| 9 | 228.912 | 600 | 229.512 | 0 | 0.28 | 0.00636 | 1.415 | 0.40 | 0.102 | 229.614 | 0.0081 | 0.0155 | 81 | 1.256 | 0.008 | 0.220 | 229.614 | | | |
| 5 | 229.586 | 500 | 230.086 | 0 | 0.20 | 0.00441 | 1.223 | 0.24 | 0.076 | 230.162 | 0.0054 | 0.0068 | 81 | 0.551 | 0.007 | 0.016 | 230.162 | | | |
| 12 | 228.994 | 400 | 229.394 | 0 | 0.13 | 0.00395 | 1.513 | 0.19 | 0.117 | 229.511 | 0.0097 | 0.0163 | 81 | 1.320 | 0.01 | 0.036 | 229.511 | | | |
| 11 | 230.798 | 300 | 231.098 | 0 | 0.07 | 0.00332 | 2.406 | 0.17 | 0.295 | 231.393 | 0.0310 | 0.0204 | 83 | 1.689 | 0.015 | 0.17 | 231.393 | 231.393 | | |
| 10 | 233.182 | 300 | 233.482 | 0 | 0.07 | 0.00332 | 1.840 | 0.13 | 0.173 | 233.655 | 0.0182 | 0.0246 | 90 | 2.214 | 0.214 | 0.17 | 233.655 | 233.655 | | |
| 8 | 230.579 | 300 | 230.879 | 0 | 0.07 | 0.00332 | 2.265 | 0.16 | 0.261 | 231.140 | 0.0274 | 0.0178 | 75 | 1.335 | 0.031 | 0.001 | 231.140 | | | |
| 7 | 233.927 | 200 | 234.127 | 0 | 0.03 | 0.00233 | 2.229 | 0.07 | 0.253 | 234.380 | 0.0389 | 0.0332 | 90 | 2.988 | 0.26 | 0.25 | 234.380 | | | |
| 6 | 237.678 | 200 | 237.878 | 0 | 0.03 | 0.00233 | 2.229 | 0.07 | 0.253 | 238.131 | 0.0389 | 0.0389 | 90 | 3.501 | 0.25 | 0.25 | 238.131 | | | |
| 4 | 230.708 | 400 | 231.108 | 0 | 0.13 | 0.00385 | 1.752 | 0.22 | 0.156 | 231.264 | 0.0129 | 0.0092 | 93 | 0.856 | 0.166 | 0.01 | 231.264 | | | |
| 3 | 231.593 | 400 | 231.993 | 0 | 0.13 | 0.00385 | 1.433 | 0.18 | 0.105 | 232.098 | 0.0087 | 0.0108 | 81 | 0.875 | 0.074 | 0.002 | 232.098 | | | |
| 2 | 232.737 | 300 | 233.037 | 0 | 0.07 | 0.00332 | 1.700 | 0.12 | 0.147 | 233.184 | 0.0154 | 0.0121 | 80 | 0.968 | 0.074 | 0.013 | 233.184 | | | |
| 1 | 234.551 | 200 | 234.751 | 0 | 0.03 | 0.00283 | 1.592 | 0.05 | 0.129 | 234.880 | 0.0198 | 0.0176 | 90 | 1.584 | 0.13 | 0.13 | 234.880 | | | |

$$n = \text{Variable} \quad S_i = K \left(\frac{V^2}{2g} \right) R^{4/3} \quad K = 2g(n^2)$$

Detailed Hydraulic Calculations for Step No 10 in Minor System Design

$$\text{M.H. 16} \quad \theta = 45^\circ$$

From Figure 4.13 $K = 0.3$

$$\therefore H_b = K \left(\frac{V^2}{2g} \right) = 0.3 \times 0.11 = 0.033\text{m}$$

$$H_t = 0.2 \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right) = 0.2 \times (0.19 - 0.11) = 0.016\text{m}$$

$$\text{M.H. 15} \quad H_t = 0.2 \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right) = 0.2 \times (0.30 - 0.19) = 0.022\text{m}$$

$$\text{M.H. 14} \quad K_b = 0.25 \quad \sqrt{\frac{10}{90}} = 0.083$$

$$H_b = K_b \left(\frac{V^2}{2g} \right) = 0.083 \times 0.30 = 0.025\text{m}$$

$$H_m = 0.05 \left(\frac{V^2}{2g} \right) = 0.05 \times 0.30 = 0.015\text{m}$$

$$\text{M.H. 13} \quad K_b = 0.25 \quad \sqrt{\frac{20}{90}} = 0.117$$

$$H_b = K_b \left(\frac{V^2}{2g} \right) = 0.117 \times 0.29 = 0.034\text{m}$$

$$(H_j + D_1 - D_2) \left(\frac{A_1 + A_2}{2} \right) = \frac{Q_2^2}{gA_2} - \frac{Q_1^2}{gA_1} - \frac{Q_3^2}{gA_3} \cos \theta$$

$$\because \theta = 90^\circ \quad \cos 90^\circ = 0$$

$$(H_j + 0.6 - 0.6) \left(\frac{0.28 + 0.28}{2} \right) = \frac{(0.67)^2}{9.81(0.28)} - \frac{(0.4)^2}{9.81(0.28)}$$

$$H_j = 0.376\text{m}$$

$$\text{M.H. 9} \quad H_b = K_b \left(\frac{V^2}{2g} \right) = (0.25 \sqrt{\frac{10}{90}}) \times 0.10 = 0.008\text{m}$$

$$\because \theta = 90^\circ \quad \cos 90^\circ = 0$$

$$\therefore (H_j + 0.5 - 0.6) \left(\frac{0.20 + 0.28}{2} \right) = \frac{(0.4)^2}{9.81(0.28)} - \frac{(0.24)^2}{9.81(0.20)}$$

$$H_j = 0.220\text{m}$$

$$\text{M.H. 5} \quad \theta = 90^\circ$$

$$H_b = K_b \left(\frac{V^2}{2g} \right) = \left(0.25 \sqrt{\frac{10}{90}} \right) \times 0.08 = 0.007m$$

$$H_t = 0.2 \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right) = 0.2 (.16 - .08) = 0.016m$$

M.H. 12 $H_b = \left(0.25 \sqrt{\frac{10}{90}} \right) \times .12 = 0.010m$

$$H_t = 0.2 (.3 - .12) = 0.036m$$

M.H. 11 $H_m = 0.05 \left(\frac{V^2}{2g} \right) = 0.05 (.3) = 0.015m$

M.H. 10 $K = 1.0$
 $H_m = K \left(\frac{V^2}{2g} \right) = 1.0 (.17) = 0.17m$

M.H. 8 $H_b = \left(0.25 \sqrt{\frac{20}{90}} \right) (.26) = 0.031m$

$$H_m = 0.1 \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right) = 0.1 (.26 - .25) = 0.001m$$

M.H. 7 $\theta = 90^\circ$
From Figure 4.13 $K = 1.04$

$$H_b = 1.04 (.25) = 0.260m$$

M.H. 6 $K = 1.0$
 $H_m = K \left(\frac{V^2}{2g} \right) = 1.0 (.25) = 0.250m$

M.H. 4 $\theta = 90^\circ$
From Figure 4.13 $K = 1.04$

$$H_b = 1.04 (.16) = 0.166m$$

M.H. 3 $H_t = 0.2 \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right) = 0.2 (.15 - .10) = 0.010m$

M.H. 2 $\theta = 60^\circ$
From Figure 4.13 $K = 0.49$

$$H_b = 0.49 (.15) = 0.074m$$

$$H_t = 0.1 \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right) = 0.002m$$

M.H. 1 $K = 1.0$
 $H_m = K \left(\frac{V^2}{2g} \right) = 1.0 \times 0.13 = 0.13$

Table 5.8 Equivalent alternative n = .012

| Street | Location | | Pipe Size mm |
|--------|--------------|------------|-----------------|
| | M.H. From | M.H. To | |
| 1 | 2 | 200 | |
| 2 | 3 | 300 | |
| 3 | 4 | 400 | |
| 4 | 5 | 400 | |
| 6 | 7 | 200 | |
| 7 | 8 | 200 | |
| 8 | 9 | 300 | |
| 10 | 11 | 300 | |
| 11 | 12 | 300 | |
| 12 | 13 | 400 | |
| 5 | 9 | 500 | |
| 9 | 13 | 600 | |
| 13 | 14 | 600 | |
| 14 | 15 | 600 | |
| 15 | 16 | 700 | |
| 16 | Outfall | 800 | |

Major System

Various manual methods can be used to estimate the major system flows. As a preliminary estimate, designers often apply the Rational formula, using the rainfall intensity for a 100 year storm and a C factor 60 percent to 85 percent higher than what would be used for a 2-year or 5-year storm. The increase in value is basically to allow for a change in the antecedent moisture condition. Except in special circumstances, a C factor above 0.85 need not be used.

In this design example the C factor of 0.35 used for the design of the minor system will be increased to 0.60, an increase of about 70 percent. The results are shown in Table 5.9.

In cases where this method results in flows in excess of the acceptable roadway capacity, a more detailed method should be applied, such as the SCS Graphical Method or a suitable hydrological computer model.

If properly laid out the major system can tolerate the variability in flows estimated by the various methods. A minor increase in the depth of surface flow will greatly increase the capacity of the major system, without necessarily causing serious flooding. The designer must also consider the remaining overland flow accumulated at the downstream end of the development; adequate consideration must be given for its conveyance to the receiving water body. This may involve increasing the minor system and inlet capacities or providing adequate drainage swales.

Table 5.9 Major system flows for 100 year storm

| Location MH to MH | Runoff | | | Time of Concentration min. | Intensity I (mm/hr) | Total Runoff Q (m³/s) | Sewer* Capacity (m³/s) | Major System (overland flow) (m³/s) | Road Grade % | Surface Capacity** (m³/s) | |
|----------------------|--------------|------|-------|----------------------------------|---------------------------|-----------------------------|------------------------------|---|--------------------|---------------------------------|------|
| | Area (ha) | C | A x C | | | | | | | | |
| 1-2 | 0.74 | 0.60 | 0.44 | 10.0 | 179 | 0.22 | 0.07 | 0.15 | 2.00 | 5.15 | |
| 2-3 | 1.10 | 0.60 | 0.66 | 1.10 | 11.5 | 0.53 | 0.11 | 0.42 | 2.00 | 5.15 | |
| 3-4 | 1.04 | 0.60 | 0.62 | 1.72 | 12.8 | 160 | 0.76 | 0.14 | 0.63 | 2.00 | 5.15 |
| 4-5 | 0.83 | 0.60 | 0.50 | 2.22 | 14.2 | 151 | 0.93 | 0.14 | 0.79 | 1.90 | 5.09 |
| 6-7 | 1.06 | 0.60 | 0.64 | 0.64 | 10.0 | 179 | 0.31 | 0.11 | 0.21 | 2.00 | 5.15 |
| 7-8 | — | — | — | — | 11.5 | 169 | 0.00 | 0.00 | 0.00 | 2.20 | 5.41 |
| 8-9 | 1.50 | 0.60 | 0.90 | 1.54 | 13.0 | 159 | 0.67 | 0.14 | 0.53 | 2.00 | 5.15 |
| 10-11 | 1.80 | 0.60 | 1.08 | 1.08 | 10.0 | 179 | 0.53 | 0.13 | 0.41 | 1.85 | 4.96 |
| 11-12 | 0.71 | 0.60 | 0.43 | 1.51 | 11.5 | 169 | 0.70 | 0.14 | 0.57 | 2.00 | 5.15 |
| 12-13 | 0.42 | 0.60 | 0.25 | 1.76 | 12.0 | 160 | 0.78 | 0.12 | 0.66 | 2.20 | 5.41 |
| 5-9 | 0.43 | 0.60 | 0.26 | 2.48 | 15.7 | 142 | 0.98 | 0.12 | 0.85 | 2.00 | 7.79 |
| 9-13 | 0.53 | 0.60 | 0.32 | 3.62 | 17.1 | 136 | 1.36 | 0.12 | 1.24 | 2.00 | 7.79 |
| 13-14 | 2.28 | 0.60 | 1.37 | 6.75 | 18.4 | 130 | 2.41 | 0.22 | 2.19 | 2.50 | 8.78 |
| 14-15 | 0.55 | 0.60 | 0.33 | 7.08 | 20.9 | 120 | 2.34 | 0.23 | 2.11 | 2.00 | 7.79 |
| 15-16 | 2.35 | 0.60 | 1.41 | 8.49 | 23.4 | 113 | 2.65 | 0.23 | 2.43 | 0.50 | 3.96 |
| 16-Outfall | — | — | — | 8.49 | 24.0 | 112 | 2.62 | 0.23 | 2.39 | 0.50 | 3.96 |

* Assuming sufficient inlet capacity

** Refer to Figure 5.5

Foundation Drains

To establish the groundwater level, piezometer measurements over a 12 month period were taken, indicating the groundwater table would be safely below the footing elevations for the proposed buildings, minimizing the amount of inflow that can be expected into the foundation drains.

The municipal requirements include detailed lot grading control, thus further reducing the possibility of surface water entering the foundation drains. Accordingly a flow value of $7.65 \times 10^{-5} \text{ m}^3/\text{s}$ per basement is used. See the discussion on Foundation Drains in Chapter 2 of this text. For detailed calculations see Table 5.10.

Computer Models

There is a wide range of computer models now available for analyzing sewer networks. The complexity of the models varies from straightforward models which use the rational method to estimate the peak flow to comprehensive models which are based on the continuity and momentum equations and are capable of modeling surcharge, backwater, orifices, weirs and other sewer components.

Table 5.11 lists several of these models and their capabilities.



Smooth-lined CSP storm sewer being installed.

Table 5.10 Foundation drain collector design sheet

| Location | From M.H. | To M.H. | Area (ha) | Unit Density (per ha) | Total Units | Cum. Units | Flow Per Unit ($m^3/s \times 10^{-3}$) | Total Flow ($m^3/s \times 10^{-3}$) | Length (m) | Gradient % | Pipe Dia. (mm) | Capacity (m^3/s) | Velocity (m/s) |
|--------------|-----------|---------|-----------|-----------------------|-------------|------------|--|---------------------------------------|------------|------------|----------------|----------------------|----------------|
| Crescent 'G' | 1A | 2A | 1.20 | 2 | 18 | 18 | 7.65 | 1.38 | 119 | 0.98 | 200 | 0.031 | 1.12 |
| Crescent 'G' | 2A | 3A | 0.72 | 2 | 11 | 29 | 7.65 | 2.22 | 94 | 1.51 | 200 | 0.037 | 1.37 |
| Crescent 'G' | 3A | 4A | 1.49 | 2 | 22 | 51 | 7.65 | 3.90 | 152 | 0.50 | 200 | 0.022 | 0.79 |
| Crescent 'G' | 4A | 5A | 0.60 | 2 | 9 | 60 | 7.65 | 4.59 | 93 | 0.55 | 200 | 0.023 | 0.82 |
| Crescent 'G' | 5A | 6A | 1.52 | 2 | 23 | 23 | 7.65 | 1.76 | 152 | 1.39 | 200 | 0.036 | 1.28 |
| Crescent 'G' | 6A | 7A | 0.93 | 2 | 14 | 37 | 7.65 | 2.83 | 90 | 2.25 | 200 | 0.040 | 1.67 |
| Crescent 'G' | 7A | 8A | 0.58 | 2 | 9 | 46 | 7.65 | 3.52 | 105 | 1.31 | 200 | 0.035 | 1.28 |
| Street 'F' | 9A | 10A | 1.54 | 3 | 30 | 30 | 7.65 | 2.30 | 137 | 1.20 | 200 | 0.034 | 1.21 |
| Street 'F' | 10A | 11A | 0.85 | 3 | 17 | 47 | 7.65 | 3.60 | 133 | 1.20 | 200 | 0.034 | 1.21 |
| Street 'A' | 5A | 8A | 0.63 | 3 | 13 | 106 | 7.65 | 8.11 | 82 | 1.81 | 200 | 0.041 | 1.52 |
| Street 'A' | 8A | 11A | 0.51 | 3 | 10 | 116 | 7.65 | 8.87 | 75 | 4.34 | 200 | 0.063 | 2.31 |
| Street 'A' | 11A | 13A | 0.94 | 3 | 19 | 135 | 7.65 | 10.30 | 133 | 1.42 | 200 | 0.036 | 1.34 |

Source: Paul Theil Associates Ltd.

Table 5.11 Computer models – Sewer system design and analysis

| Model Characteristics | CE Storm ² | HVM Dorsch ³ | ILLUDAS ⁴ | SWMM-Extran ⁵ | SWMM-Transport ⁶ | WASSP-SIM ⁶ |
|-----------------------------|-----------------------|-------------------------|----------------------|--------------------------|-----------------------------|------------------------|
| Model Purpose: | | | | | | |
| Hydraulic Design | ● | | ● | | ● | ● |
| Evaluation/Prediction | | ● | | ● | ● | ● |
| Model Capabilities: | | | | | | |
| Pipe Sizing | ● | | | | | |
| Weirs/Overflows | | ● | ● | ● | ● | ● |
| Surcharging | | ● | | ● | | ● |
| Pumping Stations | | | | ● | ● | ● |
| Storage | | | | ● | ● | ● |
| Hydraulic Equations: | | | | | | |
| Linear Kinematic Wave | ● | | ● | | | |
| Non-Linear Kinematic Wave | | | | ● | | |
| St. Venant's – Explicit | | ● | | ● | | |
| St. Venant's – Implicit | | | | ● | ● | ● |
| Ease of Use: | | | | | | |
| High | ● | | ● | | | |
| Low | | ● | | ● | ● | ● |

REFERENCES

1. Wright, K.K., *Urban Storm Drainage Criteria Manual, Volume 1*, Wright-McLaughlin Engineers, Denver, Colorado, 1969.
2. Dept. of the Army, *CE Storm Users Manual*, Construction Engineering Research Laboratory, Champaign, Illinois, 1985.
3. *Hydrograph Volume Method of Sewer System Analysis, HVM Manual*, Dorsch Consult Limited, Federal Republic of Germany, 1987.
4. Terstriep, M.L., Stall, J.B., *Illinois Urban Drainage Area Simulator (ILLUDAS)*, Illinois State Water Survey, Bulletin 58, Urbana, Illinois, 1974.
5. Huber, W.C. Heaney, J.P. and Cunningham, B.A., *Stormwater Management Model (SWMM Version IV) Users Manual*, U.S. Environmental Protection Agency, 1986.
6. *Wallingford Storm Sewer Package (WASSP), Users Guide*, Hydraulics Research Laboratory, Wallingford, UK, 1984.