Interactive hydraulic analysis for storm drain pipe systems

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An interactive hydraulic analysis computer program has been developed to aid hydraulic engineers in designing storm drain pipe systems. The entire storm drain system is analyzed for both supercritical and subcritical flow effects. By comparing the specific force (pressure plus momentum) for each pipe reach, the hydraulic grade line (HGL) and energy grade line (EGL) for the entire system can be calculated. Using this approach, both pressure and nonpressure storm drain system hydraulics can be evaluated simultaneously. Thus, a more economic storm drain pipe system can be designed by the hydraulic engineer, with the speed and computational accuracy afforded by the computer.

Key Words: gradually varied flow profile, supercritical flow, subcritical flow, pressure plus momentum, storm drain pipe system, hydraulic jump, pressure flow, open channel flow

INTRODUCTION

A strategy for the design of the storm drain and the selection of pipe sizes is to attempt to achieve pressure flow conditions whenever possible. This strategy is especially useful in regions where the land topography has a mild gradient (approximately 0.0010 to 0.0040 ft/ft). The resulting system would be somewhat optimized in that the design consideration of the hydraulic grade line would closely conform to the maximum allowable value while providing a reasonable design for flood protection purposes.

The typical design of any storm drain system requires two basic procedures. Assuming the system has been laid out in plan, with all inlets located and the rate of inflow to each determined, the first step is to sum the rate of flow in each pipe, select all pipe sizes and calculate the friction loss in each length of pipe. The second step is to calculate the change in hydraulic grade line (ΔY) at each junction. The hydraulic grade line elevation is determined at the branch point of each junction, or inlet, and the change (ΔY) at the junction is added algebraically, working progressively upstream or downstream along each pipeline. The more practical method proceeds downstream keeping the hydraulic grade line just below the street surface but low enough to accept surface flows. The last pipeline must then be sized so the hydraulic grade line is at or above the control hydraulic gradient elevation. In general, most of the storm drain analysis proceeds upstream when the depth of flow is greater than critical depth, and proceeds downstream when the depth of flow is less than critical depth. In a pressure flow system, the analysis always proceeds upstream. Also note that the storm drain analysis proceeds upstream from the downstream control depth is the more correct method but is more time consuming.

In the subject computer program, the hydraulic analysis first proceeds upstream. Therefore, the water depth is assumed to be greater than the critical depth in each drainage reach. A second analysis is then made by calculations in the downstream direction, where the depths are less than or equal to the critical depth in each drainage reach. Finally, the pressure plus momentum values for each of the two analyses are compared to determine the EGL and HGL for the entire storm drain system. Pressure flow, nonpressure flow, hydraulic jumps, and minor losses in each drainage reach can be determined individually.

FUNDAMENTALS OF HYDRAULICS

Hydraulic grade line and energy grade line

For any point in the fluid, the summation of the elevation plus the pressure head is known as the piezometric head. The piezometric head represents the level to which liquid will rise in a piezometer. The line drawn through the top of a series of piezometer columns is known as the hydraulic grade line (HGL). The energy grade line (EGL) is determined by the sum of the HGL and the velocity head (V^2/2g) such as is shown in Fig. 1a.

Specific energy

In open channel flow, the specific energy \( S_e \), is given by

\[
S_e = \gamma \cos^2 \theta + \frac{1}{2} V^2/g
\]  

(1)
where

\[ \beta = \text{momentum correction factor} \]
\[ P_A \text{ and } P_B = \text{resultant pressures acting on section A and B, respectively} \]
\[ W = \text{equivalent weight of the fluid pressure enclosed between sections A and B} \]
\[ F_f = \text{total external forces (including friction) along the wetted boundary of the channel between section A and section B} \]
\[ \theta = \text{angle of channel slope with respect to the horizontal} \]

The pressure forces are calculated by

\[ P_A = \gamma A \delta A, \quad P_B = \gamma A \delta B \]

where

\[ \gamma = \text{the specific weight of the water} \]
\[ h = \text{the distance to the centroid of the cross section below the water surface} \]

If the difference of \( W \sin \theta - F_f \) can be neglected and \( \beta = 1 \), then equation (4) can be simplified as

\[ A \delta A + Q^2/(gA) = A \delta B + Q^2/(gA) \]

Both sums of the terms in (6) involve identical components, and can be grouped together as the specific force, \( F_s \). That is,

\[ F_s = Ah + Q^2/(gA) \]

The specific force curve (Fig. 3) is similar in some of its characteristics to the specific energy curve (Fig. 2).

**Losses**

Head losses on the storm drain system are based on Los Angeles County Road Department, Design Manual (1972), Los Angeles County Flood Control District, Design Manual: Hydraulic (1979), and Orange County Flood Control District, Design Manual: Channel Hydraulics and Structures (1972).

**Friction losses**

Friction losses for pipelike conditions are computed from Manning's equation for steady flow

\[ Q = \frac{1.486}{n} A R^{2/3} S_f^{1/2} \]

**The specific force**

Consider a steady, uniform, incompressible flow in an open channel between channel section A to section B, and apply Newton's second law of motion. The second law of motion states that the change of momentum per unit time in the body is equal to the resultant of all the external forces that are acting on the body (see Fig. 1b). Thus for a fixed control volume,

\[ Q(\beta V_0 - \beta A V_A) = P_A - P_B + W \sin \theta - F_f \]

Given the flow rate \( Q \), and cross section flow area \( A \), and for \( \cos^2 \theta = 1 \),

\[ S_e = y + Q^2/2gA^2 \]

\[ (S_e - y)A^2 = Q^2/2g = \text{constant} \]

From equation (3), it is clear that the specific energy curve of Fig. 2 has the two asymptotes of \( y = S_e \), and \( y = 0 \).

**Fig. 1.** HGL and EGL for pressure and nonpressure flow system

**Fig. 2.** The specific energy curve
losses and are usually equated with velocity head by 
\[ H_i = K \cdot H_v \]

The bend losses, \( H_b \), can be estimated as

\[ H_b = 0.25 \cdot K_b \cdot H_v \]  \hspace{1cm} (11)

where \( K_b = \sqrt{\Delta/90^\circ} \) and \( \Delta \) is the central bend angle in
degrees (see Fig. 4).

The angle-point losses, \( H_{ap} \), can be estimated as

\[ H_{ap} = K_{ap} \cdot H_v \]  \hspace{1cm} (12)

where \( K_{ap} \) is a coefficient which is experimentally
determined. The coefficient \( K_{ap} \) is assumed to be a
function of the central angle (see Fig. 5) as shown in Table 2.

**Sudden pipe-reduction (contraction) losses**

The sudden contraction of a pipe flow is shown in Fig.
6. A convenient procedure for estimating the sudden
contraction losses, \( H_{ce} \), is to assume the energy head loss to
be a function of the downstream velocity head, \( H_v2 \), by

\[ H_i = K_e \cdot H_v2 \]  \hspace{1cm} (13)

where \( K_e \) is a coefficient related to the ratio of
downstream and upstream pipe flow area \( A_2/A_1 \) given by
Table 3.

---

**Table 1. Typical Manning's friction factors**

<table>
<thead>
<tr>
<th>Conduit description</th>
<th>( n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced concrete pipe (RCP)</td>
<td>0.013</td>
</tr>
<tr>
<td>Asbestos cement pipe (ACP)</td>
<td>0.012</td>
</tr>
<tr>
<td>Corrugated metal pipe (CMP)</td>
<td>0.024</td>
</tr>
<tr>
<td>Asphalt lined CMP</td>
<td>0.015</td>
</tr>
</tbody>
</table>

where

\[ Q = \text{flow rate} \]
\[ n = \text{friction factor} \]
\[ A = \text{flow area of pipe} \]
\[ R = \text{hydraulic radius} \]
\[ S_f = \text{friction slope} \]

For storm drain design purposes, the friction factor is
assumed to be a constant (regardless of the flow rate). Typical values for the friction factor are given in Table 1. The friction losses, \( H_f \), can be estimated as

\[ H_f = S_f L \]  \hspace{1cm} (9)

where \( L \) is the length of pipe.

**Manhole losses**

Manhole structures are generally constructed along the
storm drain line in order to provide an adequate
maintenance access to the pipeline. The losses, \( H_m \), due to
the passage of flow through the manhole can be estimated as

\[ H_m = K_m \cdot H_v \]  \hspace{1cm} (10)

where

\[ K_m = \text{manhole loss coefficient} \]
\[ H_v = \text{flow velocity head} \]

In this calculation, the pipe diameter is assumed to not change at the manhole.

**Bend and angle-point losses**

Bend and angle-point losses are usually limited to
pressure flow situations since the losses evident in
properly designed open channels are typically minor. Bend and angle point losses are additive to frictional...
Table 2. Typical values of $K_a$

<table>
<thead>
<tr>
<th>Angle (degrees)</th>
<th>$K_a$</th>
<th>Angle (degrees)</th>
<th>$K_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.005</td>
<td>10</td>
<td>0.030</td>
</tr>
<tr>
<td>2</td>
<td>0.008</td>
<td>12</td>
<td>0.037</td>
</tr>
<tr>
<td>3</td>
<td>0.011</td>
<td>15</td>
<td>0.047</td>
</tr>
<tr>
<td>4</td>
<td>0.014</td>
<td>20</td>
<td>0.067</td>
</tr>
<tr>
<td>5</td>
<td>0.017</td>
<td>25</td>
<td>0.090</td>
</tr>
<tr>
<td>6</td>
<td>0.020</td>
<td>30</td>
<td>0.115</td>
</tr>
<tr>
<td>7</td>
<td>0.022</td>
<td>35</td>
<td>0.146</td>
</tr>
<tr>
<td>8</td>
<td>0.024</td>
<td>40</td>
<td>0.148</td>
</tr>
<tr>
<td>9</td>
<td>0.027</td>
<td>45</td>
<td>0.234</td>
</tr>
</tbody>
</table>

Table 3. Typical values of $K_c$

<table>
<thead>
<tr>
<th>$A_2/A_1$</th>
<th>$K_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.10</td>
<td>0.46</td>
</tr>
<tr>
<td>0.20</td>
<td>0.44</td>
</tr>
<tr>
<td>0.30</td>
<td>0.36</td>
</tr>
<tr>
<td>0.40</td>
<td>0.30</td>
</tr>
<tr>
<td>0.50</td>
<td>0.24</td>
</tr>
<tr>
<td>0.60</td>
<td>0.18</td>
</tr>
<tr>
<td>0.70</td>
<td>0.12</td>
</tr>
<tr>
<td>0.80</td>
<td>0.06</td>
</tr>
<tr>
<td>0.90</td>
<td>0.02</td>
</tr>
<tr>
<td>1.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Fig. 7. Sudden enlargement model geometry

Sudden pipe-enlargement losses

The energy head loss due to a sudden enlargement of pipe size (see Fig. 7), $H_e$, is given by

$$H_e = \frac{(V_1 - V_2)^2}{2g}$$  \hspace{1cm} (14)

where $V_1$ is the upstream flow velocity and $V_2$ is the downstream flow velocity.

Transition losses

Abrupt changes in pipe size are accomplished by high energy losses. In order to reduce the losses due to a sudden expansion or contraction, structures may be designed which provide for a smooth transition for the change in pipe size. The transition losses, $H_t$, can be estimated as a function of the change in the velocity head due to the change in pipe size by

$$H_t = \begin{cases} 
  K_t (H_{x2} - H_{x1}) & \text{for } H_{x2} > H_{x1} \\
  K_t (H_{x1} - H_{x2}) & \text{for } H_{x1} > H_{x2} 
\end{cases}$$  \hspace{1cm} (15)

where $K_t$ is the transition losses coefficient. Equation (15) is assumed to apply when the transition structure wall of convergence or divergence is less than 5.75 degrees. For an angle of convergence or divergence greater than 5.75 degrees, transition losses are computed by the empirical relationship

$$H_i = 3.5 \cdot (\tan(0.00872665 \cdot \delta))^{1.22}$$  \hspace{1cm} (16)

where $\delta$ (see Fig. 8) is the total transition angle which is equal to twice the angle of convergence or divergence.

Friction losses due to the transition structure are also included in the transition losses.

Junction losses

The junction losses due to the confluence of flows of a mainline flow with one or two lateral pipeline flows may be estimated by a pressure plus momentum analysis. For example, the City of Los Angeles’ Thompson equation relates the pressure plus momentum to the change in HGL by

$$\Delta H_{GL} = \frac{Q_2 V_2 - Q_1 V_1 \cos(\text{ang1}) - Q_3 V_3 \cos(\text{ang3}) - Q_4 V_4 \cos(\text{ang4})}{g(A_1 + A_2)/2}$$  \hspace{1cm} (17)

where

$Q_1, V_1, A_1 =$ upstream flow rate, flow velocity, and pipe area

$Q_2, V_2, A_2 =$ downstream flow rate, flow velocity, and pipe area

$Q_3, V_3 =$ lateral flow rate, and flow velocity

$Q_4, V_4 =$ lateral flow rate, and flow velocity

$\text{ang1} =$ angle of confluence between upstream and downstream pipes

$\text{ang3}, \text{ang4} =$ angles of confluence between laterals and downstream pipes

Friction losses are computed using equation (8) to estimate the friction slope for both the upstream and downstream reaches. Using the average of the two friction slopes, the friction loss, $H_f$, is computed based on the length of the junction structure. Should flows enter the junction structure through an inlet constructed at the top of the structure (see Fig. 9), an additional entrance loss may be included and can be estimated as a loss associated to a catch basin inlet, $H_{eb}$, where

$$H_{eb} = 0.20 H_e$$  \hspace{1cm} (18)

The junction losses can then be expressed as

$$H_j = H_{GL} + H_{eb} - H_{x1} - H_{x2} + H_f$$  \hspace{1cm} (19)

The above pressure-plus-momentum equation is a crude approximation of the governing integral equation. In the

Fig. 8. Transition loss model geometry
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Fig. 9. Junction loss model geometry

computer program, the manhole loss of equation (10) is also computed and used for $H_f$ whenever greater.

Catch basin losses
Inlets into the storm drain system (or catch basins) are often designed in anticipation that both the HGL and EGL coincide with the ponded water surface within the inlet. Consequently, the kinetic energy of flow (or velocity head) and any losses due to the entrance of the flow into the pipe must be accounted for in the computation of the EGL within the basin. The entrance losses can be estimated as

$$H_{cb} = K_{cb} \cdot H_e$$ (26)

where $K_{cb}$ is an entrance-losses coefficient which is experimentally determined. In the program, $K_{cb} = 0.20$.

PROFILE CALCULATION

Nonpressure flow in drainage reach
In nonpressure flow systems the gradually varied flow profiles are generally computed by using any of three popular methods, namely, the graphical-integration method, the direct-integration method, and the standard step method. The standard step method continues to be the most commonly used.

In the standard step method, the computation of the flow depth is carried out on a station basis where the hydraulic characteristics are known. The computation procedure is a trial and error method to balance the energy equation.

For convenience, the position of the water surface is measured with respect to a horizontal datum. The water surface elevations above the datum at the two end sections can be expressed as (Fig. 1b)

$$Z_A = y_A + z_A$$ (21)

and

$$Z_B = y_B + z_B$$ (22)

The friction losses are estimated between points A and B by

$$h_f = S_f \cdot dx = (S_A + S_B) \cdot dx/2$$ (23)

where $S_f$ can be taken as the average of the friction slopes at the two end sections. The total head at sections A and B can be equated by the energy equation

$$S_0 \cdot dx + y_A + z_A V_A^2/2g = y_B + z_B V_B^2/2g + S_f \cdot dx + h_e$$ (24)

By substitution, the following is written

$$Z_A + c_A V_A^2/2g = Z_B + c_B V_B^2/2g + h_f + h_e$$ (25)

where $h_e$ is the eddy loss defined by

$$h_e = k(AV^2/2g)$$

where

$k = 0$ to 0.1 for gradually converging reaches
$k = 0$ to 0.2 for gradually diverging reaches
$k = 0.5$ for abrupt expansion and contraction
$k = 0$ for prismatic and regular channel

The total heads at the two end sections A and B are

$$H_A = Z_A + c_A V_A^2/2g$$ (26)

and

$$H_B = Z_B + c_B V_B^2/2g$$ (27)

Using equations (26) and (27), equation (25) can be expressed as

$$H_f = H_A + h_f$$ (28)

Given the values of $H_f$ (or $H_B$), the energy head for $H_B$ (or $H_A$) is computed by estimating possible flow depths until the governing energy equations are satisfied.

Pressure flow in drainage reach
In a pressure flow system, the calculations proceed upstream. The EGL for the upstream point of the study reach can be estimated by adding the proper head losses to the downstream EGL values. The HGL for the upstream section of the study reach is computed by subtracting the velocity head $H_v$ from the EGL, i.e., $HGL = EGL - H_v$.

Flow sealed or unsealed in drainage reach
Flow may seal or unseal in any drainage reach. If the design pipe slope is steeper than the hydraulic gradient for the conduit selected, the conduit may seal. If the design pipe slope is milder than the hydraulic gradient for the conduit selected, the conduit may seal. In both cases, sufficient pipe length must exist in order for flow to seal or unseal at a downstream section. Should flow seal in a drainage reach (Fig. 10a), the length of pipe under pressure can be estimated by

$$L = \frac{y_2 - D}{S_0 - S_f}$$ (29)
where

\[ y_2 \text{ = pressure head at downstream section} \]
\[ D \text{ = diameter of pipe} \]
\[ S_0 \text{ = designed slope of reach} \]
\[ S_f \text{ = friction slope of reach} \]

Should flow unseal in a drainage reach (Fig. 10b), the gradually varied flow profile proceeds until the depth reach the pipe diameter. Thereafter, the pressure flow friction losses is used to estimate the HGL and EGL for the upstream section.

**Hydraulic jump in drainage reach**

A hydraulic jump in a drainage reach can occur only when upstream flow is in a supercritical flow regime and the downstream flow is in a subcritical flow regime. Both the upstream and downstream hydraulic analysis should be performed in order to approximate the gradually varied flow profile for this drainage reach.

**HGL after head losses**

At pipe enlargement or reduction locations, the pipe sizes change. The new water depth (nonpressure flow) or the new pressure head (pressure flow) should be adjusted according to the changes of pipe size. After changing in pipe size, the specific energy can be estimated for the new pipe sizes. Then, a new specific energy curve is constructed so that the new water depth or pressure head can be determined with respect to the new specific energy. Notice that the water depth cannot be greater than the critical depth when the flow analysis proceeds downstream, and the water depth or pressure head cannot be less than the critical depth when the flow analysis proceeds upstream. When pipe sizes remain constant, the above procedure should be followed without constructing a new specific energy curve. For head losses which depend upon both upstream and downstream velocity head, an iteration procedure is used to balance the head losses with respect to the upstream and downstream velocity head.

**Profile determination**

For each drainage reach, the pressure plus momentum values are calculated for both upstream and downstream analyses at downstream and upstream sections. Higher pressure plus momentum values will be used to determine the water surface profile (see Table 4).

**STORM DRAIN COMPUTER MODEL**

A storm drain computer model based on the storm drain pressure flow model\(^{9}\) was developed to illustrate the hydraulic analysis procedures. This program employed the user-friendly, form fill-out data technique\(^{1}\) to increase the user efficiency, and decrease the total cost of engineering design process. The storm drain computer model is composed of a Main Menu program and nine subroutine analysis procedures. The model is developed by linking the Main Menu selection program to each subroutine in order to enable the engineer to branch to the desired analysis procedure when optioned. The various energy loss calculation options are listed in Table 5.

---

**Table 4. Logic of profile determination**

<table>
<thead>
<tr>
<th>Pressure plus momentum</th>
<th>Upstream analysis</th>
<th>Downstream analysis</th>
<th>Flow regime</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&gt;</td>
<td>&gt;</td>
<td>Subcritical flow</td>
</tr>
<tr>
<td>Upstream section</td>
<td>≥</td>
<td>≥</td>
<td></td>
</tr>
<tr>
<td>Upstream section</td>
<td>≤</td>
<td>≤</td>
<td>Supercritical flow</td>
</tr>
<tr>
<td>Downstream section</td>
<td>≤</td>
<td>≤</td>
<td></td>
</tr>
<tr>
<td>Upstream section</td>
<td>≥</td>
<td>≥</td>
<td>Hydraulic jump</td>
</tr>
<tr>
<td>Downstream section</td>
<td>≤</td>
<td>≤</td>
<td></td>
</tr>
</tbody>
</table>

**Table 5. Storm drain computer model programs**

<table>
<thead>
<tr>
<th>Program</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Main menu</td>
</tr>
<tr>
<td>2</td>
<td>Friction losses</td>
</tr>
<tr>
<td>3</td>
<td>Manhole losses</td>
</tr>
<tr>
<td>4</td>
<td>Bend losses</td>
</tr>
<tr>
<td>5</td>
<td>Sudden enlargement losses</td>
</tr>
<tr>
<td>6</td>
<td>Junction losses</td>
</tr>
<tr>
<td>7</td>
<td>Angle point losses</td>
</tr>
<tr>
<td>8</td>
<td>Sudden contraction losses</td>
</tr>
<tr>
<td>9</td>
<td>Catch basin losses</td>
</tr>
<tr>
<td>10</td>
<td>Transition losses</td>
</tr>
</tbody>
</table>

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Data entry for various programs are depicted as follows:

PROGRAM 1: DATA ENTRY

Program 1: Data Entry

--- Data Entry for Pipe Losses ---

Enter pressure drop (inches)
--- INCH ---
--- Allowable Values are 0.0 to 2.0 ---

Enter pipe diameter (inches)
--- INCH ---
--- Allowable Values are 3.0 to 12.0 ---

Enter pipe length (feet)
--- FEET ---
--- Allowable Values are 2.0 to 100.0 ---

Enter Manning’s friction factor
--- INCH ---
--- Allowable Values are 0.0 to 1.0 ---

--- Total: 55 to leave program; 777 to go to top of page ---

PROGRAM 2: DATA ENTRY

Program 2: Data Entry

--- Data Entry for Friction Losses ---

Enter pressure drop (inches)
--- INCH ---
--- Allowable Values are 0.1 to 2.0 ---

Enter pipe diameter (inches)
--- INCH ---
--- Allowable Values are 3.0 to 12.0 ---

Enter length of pipe (feet)
--- FEET ---
--- Allowable Values are 2.0 to 100.0 ---

Enter Manning’s friction factor
--- INCH ---
--- Allowable Values are 0.0 to 1.0 ---

--- Total: 55 to leave program; 777 to go to top of page ---

PROGRAM 3: DATA ENTRY

Program 3: Data Entry

--- Data Entry for Friction Losses ---

Enter pressure drop (inches)
--- INCH ---
--- Allowable Values are 0.1 to 2.0 ---

Enter pipe diameter (inches)
--- INCH ---
--- Allowable Values are 3.0 to 12.0 ---

Enter length of pipe (feet)
--- FEET ---
--- Allowable Values are 2.0 to 100.0 ---

--- Total: 55 to leave program; 777 to go to top of page ---

PROGRAM 4: DATA ENTRY

Program 4: Data Entry

--- Data Entry for Pipe Losses ---

Enter pressure drop (inches)
--- INCH ---
--- Allowable Values are 0.0 to 2.0 ---

Enter pipe diameter (inches)
--- INCH ---
--- Allowable Values are 3.0 to 12.0 ---

Enter pipe length (feet)
--- FEET ---
--- Allowable Values are 2.0 to 100.0 ---

Enter Manning’s friction factor
--- INCH ---
--- Allowable Values are 0.0 to 1.0 ---

--- Total: 55 to leave program; 777 to go to top of page ---

PROGRAM 5: DATA ENTRY

Program 5: Data Entry

--- Data Entry for Friction Losses ---

Enter pressure drop (inches)
--- INCH ---
--- Allowable Values are 0.1 to 2.0 ---

Enter pipe diameter (inches)
--- INCH ---
--- Allowable Values are 3.0 to 12.0 ---

Enter length of pipe (feet)
--- FEET ---
--- Allowable Values are 2.0 to 100.0 ---

Enter Manning’s friction factor
--- INCH ---
--- Allowable Values are 0.0 to 1.0 ---

--- Total: 55 to leave program; 777 to go to top of page ---

PROGRAM 6: DATA ENTRY

Program 6: Data Entry

--- Data Entry for Friction Losses ---

Enter pressure drop (inches)
--- INCH ---
--- Allowable Values are 0.1 to 2.0 ---

Enter pipe diameter (inches)
--- INCH ---
--- Allowable Values are 3.0 to 12.0 ---

Enter length of pipe (feet)
--- FEET ---
--- Allowable Values are 2.0 to 100.0 ---

Enter Manning’s friction factor
--- INCH ---
--- Allowable Values are 0.0 to 1.0 ---

--- Total: 55 to leave program; 777 to go to top of page ---
APPLICATION

An example problem taken from the Highway Design Manual of Instruction (the Los Angeles Road Department) is used to illustrate the capability of the storm drain computer model. The analysed drainage system is depicted on Fig. 11. The downstream hydraulic grade line is assumed to be at elevation 196.70. The upstream sections 14 and 15 are the transition structure and box structure respectively. The box structures were not analysed because of the limitation of the model (circular section only). Therefore, the upstream control depth was assumed to be the normal depth (1.61 ft) of that reach. In the reach between sections 12 and 14, the water surface was defaulted to normal depth due to the steep slope. A flow depth of 1.84 ft was estimated after flow
Fig. 11. Hydraulics example

Fig. 11. Hydraulics example (continued)
passing through a manhole structure at section 12. A hydraulic jump was predicted by the storm drain computer model and the location of the pressure plus momentum balance occurred about 255.48 ft from section 12. A gradually varied flow profile was also calculated by the model for this nonpressure segment. The example calculated in the manual used the normal depth (1.69 ft) for the nonpressure flow segment. The normal depth provided less pressure plus momentum forces to push the flow downstream (27.15 ft) as model predicted (322.8 ft). Pressure system calculations were well-predicted by the model when compared to the manual results from section 11 to section 1.

Computer model results are included in the Appendix. First the nodal point status table which contains the flow depth, pressure head and pressure plus momentum for upstream and downstream analyses is printed for user's convenience. The section (node) numbers are arranged from upstream to downstream. User specified head loss options are also printed. 'Hydraulic jump' will be printed when it occurred in the pipe reach. Control pressure head on flow depth of each section is followed by an asterisk for user's convenience. Entire hydraulic analysis is also included after the nodal point status table. Head loss calculations, HGL, EGL and flow line are printed for each section. Only one gradually varied flow profile will be selected with respect to the control flow depth. In a reach where hydraulic jump occurs, both supercritical and subcritical flow profiles are printed. However, the determination of the location and length of hydraulic jumps is not included in the programming. Rather, this type of information is currently indeterminate and is left to the engineer for special consideration on a case by case basis. A common approach is to assume the jump to occur as a shock whereby the conjugate depths are matched at a single point, with the length of the jump being assumed as zero. The type of solution may be unacceptable in cases where a pipe lateral enters the main channel immediately upstream of such an assumed hydraulic jump shock, and the hydraulic control for the pipe is assumed to be the lower conjugate depth.

DISCUSSIONS

The storm drain computer model has the capability to analyse a general storm drain pipe system. Furthermore, it can analyse hydraulic jumps, pressure and nonpressure flow in any drainage reach. Gradually varied flow profiles are approximated by the standard step method when nonpressure flow occurs in any pipe reach. This analysis provided the hydraulic engineer a better understanding of the storm drain system hydraulics when pressure and nonpressure flow co-exist in the storm drain system.

Because the computer program is interactive, the pipe system can be quickly designed without the use of a data batch-file approach.

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APPENDIX: EXAMPLE RESULTS