Storm Sewer Junction Hydraulics and Sediment Transport

Stuart M. Stein, P.E., Xibing Dou, Ph.D., Edward R. Umbrell, J. Sterling Jones, P.E.

Introduction

Stormwater can carry substantial loadings of suspended solids into storm sewer systems. A potentially important issue associated with sediment transport through sewers is "shock loading." This occurs when periods of low flow leave significant sediment deposits in sewers and manholes. A large event can then provide the energy to scour the deposited material and transport far more loading to a receiving waterbody than is generated from the land surface during the event. This could be a potentially critical issue in developing TMDLs since loading might be far greater than expected for large events, and lower than expected for small events. This paper will present the results of significant physical modeling research on energy losses associated with manholes and discuss how these losses might effect sediment transport. Additionally, a 3-D hydrodynamic model was applied to various manhole configurations to qualitatively assess circulation tendencies and sediment movement within manholes.

Physical Modeling of Storm Sewer Junction Hydraulics

While head losses can not be directly correlated to sediment transport potential, these losses are caused by the high turbulence which exists in a manhole in high flow conditions. The water churns and bubbles violently (secondary motion), even under steady-state inflow conditions. This high turbulence has the potential to suspend particles which had previously settled to the manhole floor. So even if the horizontal velocity in the junction is relatively small (remember that water elevations in a manhole can be far higher that the crowns of inflow and outflow pipes), particles may be continuously resuspended and available for outflow transport.

The hydraulic analysis through a manhole focuses on the calculation of the energy loss from the inflow pipes to the outflow pipe. Several determining factors affect the computation of the energy loss coefficient in the HGL methodology. These include the manhole size relative to the outlet pipe diameter, the depth of flow in the manhole, the amount of discharge, the inflow pipe angle, the plunge height, the relative pipe diameter, and the floor configuration.

A lab study was performed in an attempt to isolate these different factors. A test matrix was developed to examine the seemingly endless number of physical configurations. Hundreds of runs were conducted under a wide variety of flow conditions. Thousands of data points were collected and analyzed. The hydraulic gradeline for the flow was measured and from that, the energy gradeline was calculated. The energy loss though the manholes was calculated and the results were used to create the empirical equations presented in this paper (FHWA, 1996). Needless to say, the results tended to exhibit a lot of scatter.

The hydraulic gradeline methodology starts with equation (1) which describes the energy loss for an inflow pipe:

$$\Delta E = \left(C_1 \, C_2 \, C_3 + C_{4_i}\right) w \, \frac{V_o^2}{2g} \tag{1}$$

Where:

ΔE	=	Energy loss for an inflow pipe, m.
C_1	=	Coefficient related to relative manhole size.
C_2	=	Coefficient related to water depth in the manhole.
C_3	=	Coefficient related to lateral flow, lateral angle, and plunging flow.
C_{4_i}	=	Coefficient related to relative pipe diameters.
ω	=	Correction factor for benching.
$\frac{V_o^2}{2g}$	=	Velocity head, m/s^2.

<u>Relative manhole size</u>

The larger the manhole is relative to the outlet pipe, the greater the space for the flow to expand and dissipate the velocity head. Similarly, the greater the expansion into the manhole, the greater the energy losses in contracting to leave through the outlet pipe. The modeling indicates that for b/D_0, (manhole diameter/outlet pipe diameter), values up to 4.0, the coefficient related to manhole size, C_1, is calculated with the following equation:

$$C_1 = \frac{0.9\left[\frac{b}{D_0}\right]}{\left[6 + \frac{b}{D_0}\right]} \tag{2}$$

Where:

b	=	Manhole diameter, m.
D_0	=	Outflow pipe diameter, m.

Once the manhole diameter is four times the outlet pipe diameter, or larger, the manhole is "large" and the coefficient C_1 is assumed to be a constant equal to 0.36.

Water Depth in the Manhole

The coefficient, C_2, which is related to manhole water depth, increases rapidly with relative water depth, d_{mH}/D_o, up to 2.0. The rate of the increase slows when d_{mH}/D_0 reaches approximately 3.0. This type of curve can be expressed as a third order polynomial. Equation (3) applies for $d_{mH}/D_0 \leq 3.0$. When d_{mH}/D_0 is greater than 3.0, C_2 is equal to 0.82. The following equation was found to fit the data reasonably well:

$$C_2 = 0.24\left(\frac{d_{mH}}{D_0}\right)^2 - 0.05\left(\frac{d_{mH}}{D_0}\right)^3 \qquad (3)$$

Where:

d_{mH} = Depth in the manhole relative to the outlet pipe invert, m.

Multiple inflows

The coefficient related to multiple inflows, C_3, is the most complex term in the composite energy loss coefficient equation. The effect of lateral flows on the energy loss was studied with respect to three parameters: flow rate, connecting angle of the inflow pipe, and elevation of the inflow pipe. The following equation can be used to calculate the coefficient C_3.

$$C_3 = \text{Term 1} + \text{Term 2} + \text{Term 3} + \text{Term 4} + \text{Term 5} \qquad (4)$$

Where:

Term 1 = 1.0

Term 2 = $\sum_{i=1}^{4}\left(\frac{Q_i}{Q_0}\right)^{0.75}\left[1 + 2\left(\frac{Z_i}{D_0} - \frac{d_{mH}}{D_0}\right)^{0.3}\left(\frac{Z_i}{D_0}\right)^{0.3}\right]$

Term 3 = $4\sum_{i-1}^{3}\frac{(\mathrm{C}os q_{\mathrm{i}})(HMC_i)}{\left(\frac{d_{mH}}{D_0}\right)^{0.3}}$

Term 4 = $0.8\left|\frac{Z_a}{D_0} - \frac{Z_B}{D_0}\right|$

Term 5 = $\left|\left(\frac{Q_A}{Q_0}\right)^{0.75} Sin q_A + \left(\frac{Q_B}{Q_0}\right)^{0.75} Sin q_B\right|$

$$HMC_i = \left[0.85 - \left(\frac{Z_i}{D_0}\right)\left(\frac{Q_i}{Q_0}\right)^{0.75}\right] \qquad (5)$$

and:

Q_0	=	Total discharge in the outlet pipe, m^3/s.
Q_1, Q_2, Q_3	=	Pipe discharge in inflow pipes 1, 2 ,and 3, m^3/s.
Q_4	=	Discharge into manhole from the inlet, m^3/s.
Z_1, Z_2, Z_3	=	Invert elevation of inflow pipes 1, 2, and 3 relative to the outlet
Z_4	=	Elevation of the inlet relative to the outlet pipe invert, m.
D_0	=	Outlet pipe diameter, m.
b	=	Manhole diameter, m.
d_{mH}	=	Depth in the manhole relative to the outlet pipe invert, m.
q_1, q_2, q_3	=	Angle between the outlet main and inflow pipes 1, 2, and 3, degrees.
HMC_i	=	Horizontal momentum check for pipe i.
$Q_A, Q_{\mathbf{B}}$	=	Pipe discharges for the pair of inflow pipes that produce the largest value for term 4, m^3/s.
$Z_A, Z_{\mathbf{B}}$	=	Invert elevation, relative to outlet pipe invert, for the inflow pipes that produce the largest value for term 4, m.

All angles are represented between 0 and 360 degrees for this equation and are measured clockwise from the outlet pipe. For a simple two-pipe system with no plunging flow, C_3 is assumed to be equal to 1.0.

The second term in equation (4) captures the energy losses from plunging inflows. This term reflects the fact that flows plunging from greater heights result in greater turbulence and, therefore, higher energy losses. As shown by the summation in the second term, the computation is valid for one to three inflow pipes and plunging flow from the inlet.

The third term reflects the effects the angle (with respect to the outflow pipe) has on energy losses. If the horizontal momentum check (HMC_i) is less than 0, the flow is falling from a height such that the horizontal momentum is assumed to be negligible and term 3 is set to 0.

The fourth and fifth terms deal with lateral flows and what effect, if any, they have on the overall energy losses.

Relative pipe diameters

Although no experiments were performed with different pipe diameters in this study, a correction for such a case is required in the hydraulic gradeline analysis. Equation (6) was theoretically derived based on conservation of momentum and is proposed for this purpose. Each loss is unique to each inflow pipe and does not affect any other inflow pipes.

$$C_{4_i} = 1 + \left[\left(\frac{Q_i}{Q_o} + 2\frac{A_i}{A_o}\cos\theta_i \right) \frac{V_i^2}{V_o^2} \right] \qquad (6)$$

Where:

A_i,A_o = Cross-sectional area of inflow and outflow pipes, m^2.
θ_i = Angle between outflow pipe and inflow pipe i, degrees.

If θ_i for any pipe is less than 90 degrees or greater than 270 degrees, ($\cos \theta_i$) is replaced with 0 in equation (6). This sets the maximum exit loss to be the incoming velocity head.

Floor configuration

Manhole benching affects head loss because the bottom channels help guide the flow smoothly into the outlet pipe reducing turbulence in the manhole. The reduction in the head loss is dictated by the type and extent of benching. There are three types of benchings that are commonly used in drainage practice. One type is a half benching for which the lower half of the pipe extends through the junction and horizontal benches are extended from the semi-circular channel to the junction wall. The second type is a full benching which is an improved variation on the half benching obtained by extending the mold side wall to the pipe crown elevation. The third type is an improved variation of the full benching which adds smooth transition sections to the inflow and outflow pipes.

The correction factors, ω, shown in table 1, should be multiplied by the head-loss coefficient for a manhole with a flat floor. For conditions that are between clear pressure flow ($d_{mH} / D_0 > 3.2$) and clear free-surface flow ($d_{mH} / D_0 < 1.0$), a linear interpolation is an appropriate approximation.

Table 1. Correction factors, ω, for benching

	Bench Submerged*	Bench Unsubmerged**
Flat Floor	1.00	1.00
Benched one-half pipe diameter high	0.95	0.15
Benched one pipe diameter high	0.75	0.07
Improved	0.40	0.02

* pressure flow, $d_{mH} / D_0 > 3.2$
** free-surface flow, $d_{mH} / D_0 < 1.0$

Three-Dimensional Hydrodynamic Modeling

In order to simulate the complex turbulent flow field, a Stochastic Turbulence-Closure Model that predicts anisotropic turbulent stresses was applied. This 3-D turbulence Reynolds Stress Model was first proposed by Dou (Dou, 1980). In this model, the anisotropic turbulence stresses are computed directly from the fluctuating velocity correlations derived by Dou by using a stochastic approach as follows:

$$-\overline{u_i' u_j'} = -\left(\frac{1}{2}U^2 + 2M^2L^2\frac{\partial U_i}{\partial x_m} + \frac{\partial U_j}{\partial x_l}\right)\delta_{ij}$$

$$+\frac{1}{2}\left[U^2T - \frac{1}{2}ML^2\left(\frac{\partial U_i}{2x_m} + \frac{\partial U_j}{2x_l}\right)\right]\left(\frac{\partial U_i}{2x_j} + \frac{\partial U_j}{x_i}\right)$$

$$-\frac{1}{8}M^2L^2\left(3\delta_{lm} - 1\right)\left(\frac{\partial U_i}{2x_m} + \frac{\partial U_m}{2x_i}\right)\left(\frac{\partial U_j}{2x_l} + \frac{\partial U_l}{2x_j}\right) \qquad (6)$$

where ***U***, ***L*** and ***T*** are the characteristic velocity, length, and time. The subscripts *m* and *l* are the summation indices (= 1, 2, 3). δ_{ij} and δ_{lm} are the Kronecker deltas. *M* is a dimensionless parameter that represents viscous effects. This model 3-D model is currently being applied by GKY&A for the Federal Highway Administration (FHWA). It is also linked to a sediment transport model which has been implemented for bridge scour but not storm drain sediment movement.

Figures 1 and 2 show the plan view for velocity vectors through a manhole with inflow and outflow pipes at 180^{o}, at low depth (below the inflow pipe invert) and a high depth (above the inflow pipe invert, respectively. Figure 1 indicates that horizontal velocity is extremely low below the invert for this configuration. Figure 2 shows that the flow travels straight across without much expansion at the higher elevation. In other words, the 180^{o} configuration does not experience much expansion and contraction which translates to head loss.

Figures 3 and 4 show the same information as did 1 and 2 for a manhole configuration with pipes at 90^{o}. At the lower depth, much of the pipe has developed a counterclockwise flow pattern. At the higher depth, the flow shoots through the manhole from the inflow pipe and crashes against the far side. These figures indicate that most of the velocity head is lost for this configuration, which is consistent with our laboratory experiments.

Conclusions

Estimating sediment transport through storm sewer junctions is extremely complex. Our physical and numerical modeling of sewer indicates that the hydraulics of complex junctions is not yet fully understood. Only after the complex 3-dimensional hydraulics is better quantified can precise models of sediment transport be developed. In the meantime, the numerical modeling can qualitatively indicate the losses (and turbulence) associated with various manhole configurations. The turbulence might then be used as an indicator of sediment resuspension which can affect sediment loads during events.

References

Dou, Guoren (1980), “The stochastic theory and the general law of all flow regions for turbulent open channel flows,” Proceeding of First International Symposium on River Sedimentation, Beijing, China.

FHWA (1996), “HYDRAIN-Integrated Drainage Design Computer System: Version 6.0,” Technical Report No. FHWA-SA-96-064, Office of Technology Application, Office of Engineering, Federal Highway Administration, Washington, DC 20590.

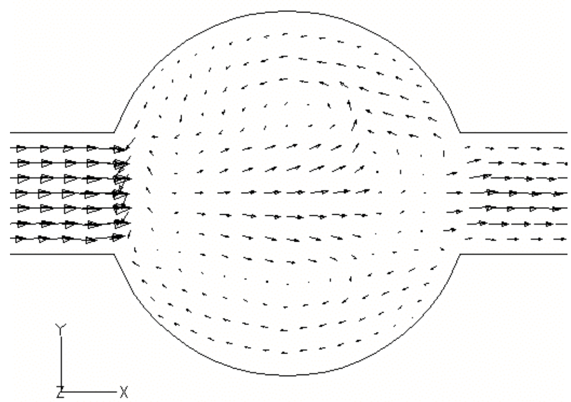

Figure 1. Velocity vectors at low depth (180° configuration)

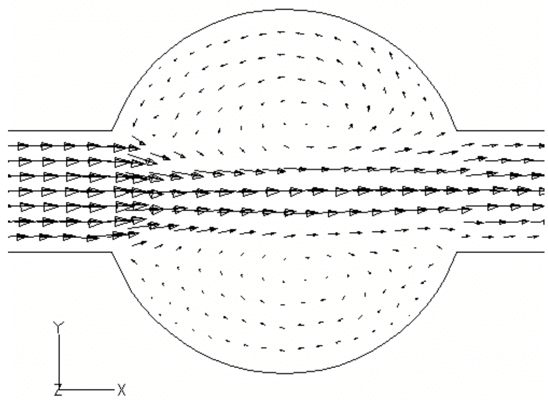

Figure 2. Velocity vectors at high depth (180° configuration)

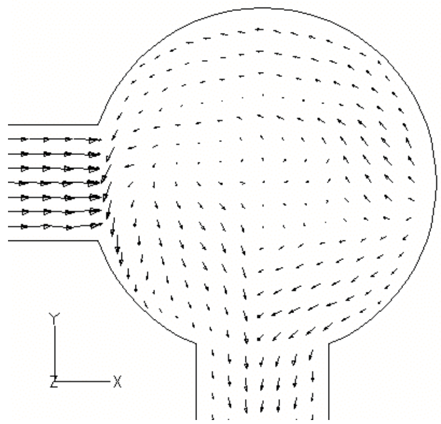

Figure 3. Velocity vectors at low depth (90° configuration)

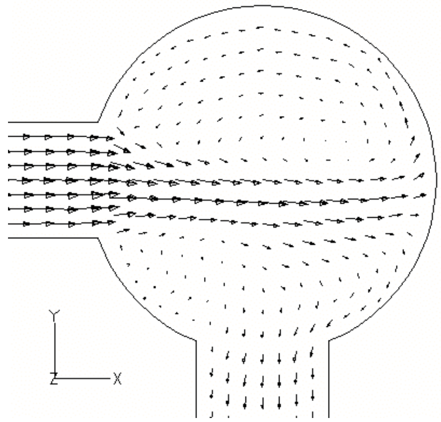

Figure 4. Velocity vectors at high depth (90° configuration)