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Head Loss at Manholes in Surcharged Sewers

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Abstract

Rehabilitated sewer systems in Houston are being designed to operate for short periods in a surcharged condition. The operational evaluation of how the system will function is based on hydraulic models that currently neglect junction losses. These losses are indeed negligible in gravity flow, but recent physical modeling experiments at the University of Houston indicate that the losses in pressure flow are significant and amount to a loss equivalent to several hundred feet of pipeline. For a handful of junctions, this added loss is small, but in a system with hundreds of junctions and miles of pipelines these losses will have an impact on the prediction of system performance.

Our results show that junction losses are most significant when the junction changes the flow direction or blends flows from different directions. The results imply that sewer hydraulics models should consider junction losses when the number of junctions is large. The head loss results are reported as minor loss coefficients and as equivalent pipe lengths. The use of equivalent pipe length should allow the inclusion of junction losses into models that were not originally designed to directly include junction losses.

Introduction

Head loss through sewer junctions is often neglected when the sewers are flowing normally (gravity flow). Typical formulas for estimating loss through a gravity flow junction (American Iron and Steel Institute, 1990) support the assumption of negligible head loss through junctions during normal flow. However, when the system is surcharged (pressure flow) junction losses may become important, especially in large systems with many junctions.

Rehabilitated sewer systems in Houston are being designed to operate intentionally surcharged for short periods. The operational evaluation of how the system will function is based on hydraulic models that currently assume negligible junction losses because there is very little experimental and theoretical support for calculating the magnitude of these losses.

Maraslek (1985) studied head losses in pressurized sewer junctions and reported that the most important variables included relative lateral inflows for junctions with two or more pipes. The losses were observed to increase as the ratio of lateral inflow to main line inflow increased. His study also reported that relative pipe sizes, benching, and pipe alignment were more important than base shape for determining the magnitude of head loss in the junctions. Although the study was extensive, general design values were not reported, and Maraslek, like Chow (1959) concluded that losses can best be determined by experimental analysis.

Pedersen and Mark (1990) developed a theory to calculate head loss in junctions with a straight through flow and fully submerged inlet and outlet pipes of equal diameter. Their theory predicts losses about one-order of magnitude smaller than one would predict using a traditional entrance and exit loss approach (assuming the manhole behaves as a stilling basin). Unfortunately their theory is limited to the straight-through flow case, and the authors state that "from a practical point of view, the theory itself is too complicated to be incorporated in a numerical model of sewer networks ..."

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Serre et al. (1992) studied the energy loss at a 90-degree junction connecting to a main line. They also discovered that design data for this type of calculation were either non-existent or so uncertain as to be useless. They developed a theory for a three-pipe junction based on the momentum equation and developed a semi-empirical relationship for calculating loss coefficients. While their work is a significant contribution, the case of a four-pipe junction and a two-pipe junction of arbitrary angle is left unanswered.

This paper reports a physical modeling study aimed at developing semi-empirical relationships for predicting the head loss in surcharged junctions for typical configurations in the City of Houston's system. The physical model used 2-inch to 4-inch pipes to simulate flow in 12-inch to 48-inch pipes using 1/6 and 1/8 geometric scaling and Froude number similarity.

Physical Model

The physical modeling system is about forty feet long in the main line axis and the lateral line axis. The junction model is a 1/6 scale model of a typical manhole with a main inlet, designated "M", and two lateral lines, designated "A" and "B", flowing into the model and a single outlet, designated "O" leaving the model. Each line is instrumented with manometers and a Doppler-shift ultrasonic velocity meter to measure piezometric and kinetic heads in the system. Total discharge is measured at the outlet using a calibrated weir. Figure 1 is a plan view of the modeling system.

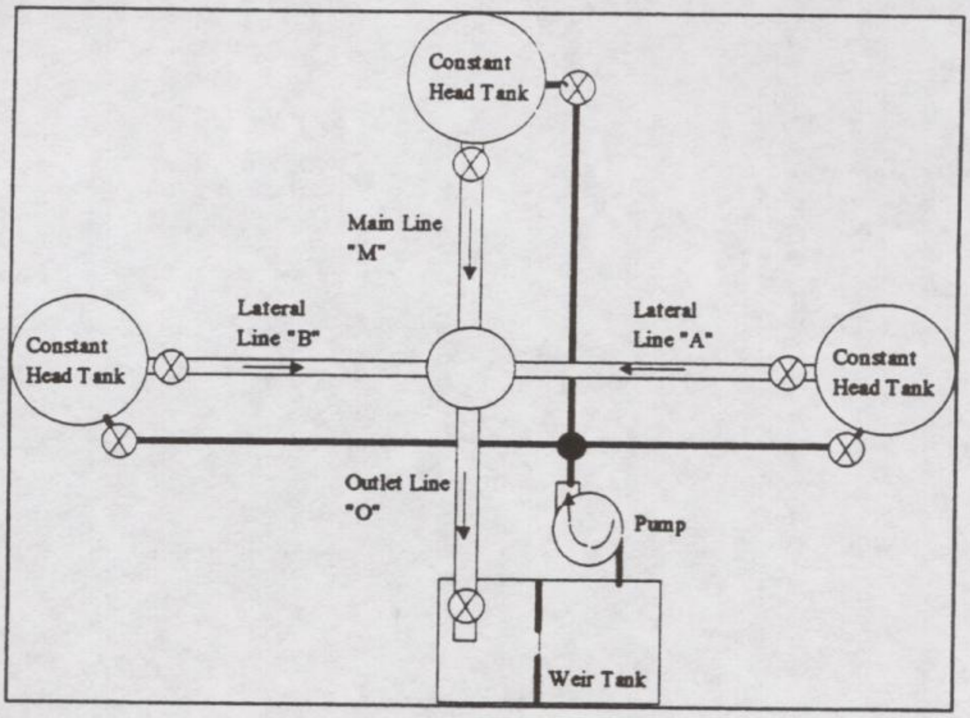

Figure 1. Modeling System Schematic (Plan View)

By blocking one or more of the inlets with plugs, or switching the manhole base model a variety of configurations with different pipe diameters can be studied. These configurations include: straight-line flow ("I"), 90-degree junction ("L"), main line with one lateral ("T"), a full cross-flow situation ("X"), and a 45-degree junction ("J"). Scaling factors based on Froude number similarity allow the model results to be scaled to real pipe dimensions in the sewer system.

The manhole model is an 8-inch cylinder, 40-inches high, with four openings for 4-inch PVC pipe around its circumference at the base. Semi-circular channels are cut to model "benching" typical of manhole specifications in use by the City. Figure 2 is a photograph of the manhole base (with the riser removed) showing four lines connecting in an "X" configuration. Adapters allow the use of 3-inch or 2-inch inlet pipes, with crown elevations matching the outlet pipe crown, as per typical design practice. The model is clear plastic so that the water flow patterns can be directly observed during experiments.

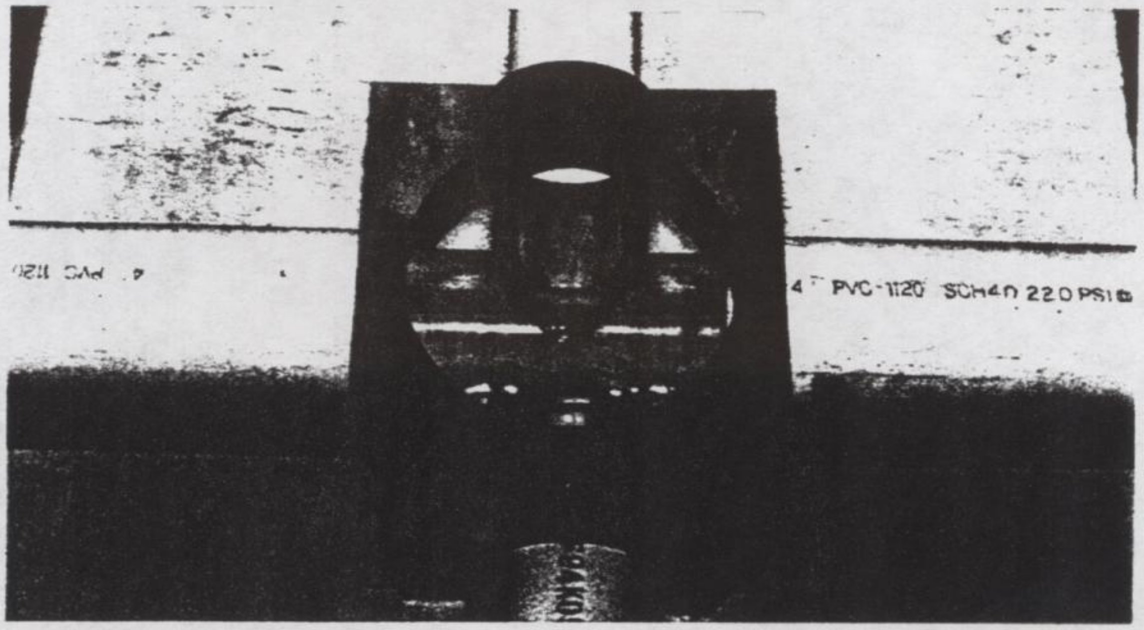

Figure 2. Inside of Model Base (outlet on right). Benched for 4-inch Main, 3-inch Laterals

Figure 3 is a photograph of the measuring tank and weir looking upstream towards the outlet pipe in the modeling system; the tank is 4-feet wide, 4-feet tall, and 8-feet long, with a 7-inch wide sharp edged, contracted weir.

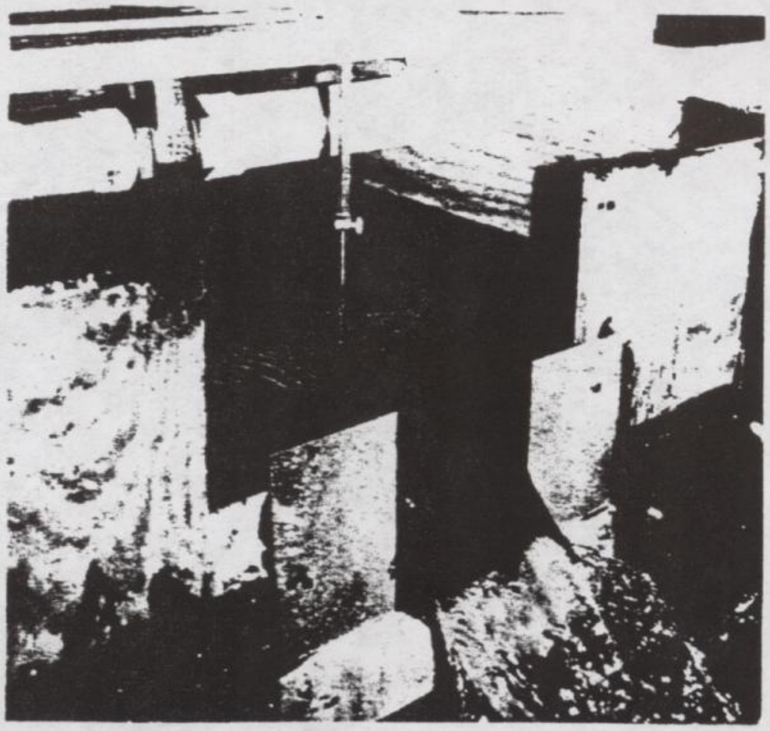

Figure 3. Measurement weir, with point gage and outlet line from model in background.

A rating curve for the weir was developed by weighing the water flowing into the tank over measured time periods. These values were plotted against depth, and fitted to standard discharge equations to determine a rating formula for subsequent measurements.

The ultrasonic Doppler-shift flowmeters are used to measure flow velocities in the inlet lines for computing the kinetic energy of water in these lines for subsequent energy loss calculations. They are calibrated by comparison to the weir flow measurement when only a single line is flowing. When all three lines are flowing the agreement of their total discharge (computed as the product of velocity and cross sectional area) with the weir measurement is typically within 2%.

The manometers are placed at 2-foot intervals on each line to determine the hydraulic grade line in any of the pipes. The first manometer port in each line is located 16 pipe diameters downstream from an inlet (to the line) to help reduce entry effects in the measurements. Unfortunately, our laboratory space was too limited to use a greater distance, and entry effects are observable in first downstream manometer from each inlet.

Energy Equation for the Modeled System

The energy loss associated with flow into and out of the manhole is analyzed using the energy equation. The energy equation along the in-line main pipe is expressed as

$$h_m + \frac{V_m^2}{2g} = h_o + \frac{V_o^2}{2g} + K_m \frac{V_o^2}{2g} \qquad (1)$$

where h_m and h_o are the piezometric heads for the inlet main line and the outlet line, respectively. V_m is the average velocity in the main pipe and V_o is the average velocity in the outlet. K_m is the head loss coefficient due to flow through the junction from the main pipe to the outlet pipe.

A similar approach for the lateral inflow lines to the outlet pipe gives the expression

$$h_a + \frac{V_a^2}{2g} = h_o + \frac{V_o^2}{2g} + K_a \frac{V_o^2}{2g} \qquad (2)$$

where h_a is the piezometric heads in the lateral, V_a is the velocity in the lateral, and K_a is the head loss coefficient due to flow through the junction from the lateral to the outlet. If a second lateral is included a similar expression incorporating the head loss coefficient K_b is included.

The head loss coefficients are determined from the measured data. Table 1 below is a typical data set for an entire experimental scheme (different flow ratios) with results scaled up using a 1:6 geometric scaling scheme. The scaling factors to change measured model values into practical (full scale values) are:

$$\lambda = \frac{l_p}{l_m}; \quad V_p = V_m\sqrt{\lambda} \quad Q_p = Q_m\sqrt{\lambda^5} \qquad (3)$$

where λ is the geometric scaling factor, l_p is the prototype (full scale) characteristic length, l_m is the model characteristic length, V_p is the prototype (full scale) velocity, Vm is the model (measured) velocity, Q_p is the prototype (full scale) discharge, and Q_m is the model (measured) discharge.

For the data of Table 1 to be put into practical use with existing models that do not directly incorporate expressions for junction losses, the equivalent added pipe length for the various pipes is included in the tables. The junction head loss is expressed as the equivalent friction loss due to added pipe length, This additional pipe length is added or subtracted from the actual pipe length in a numerical model to include the effect of junction losses in modeling.

Table 1. Manhole head loss coefficients and equivalent pipe length for different pi[illegible] configurations at various flow rates (24-inch main line and 18-inch laterals, n= 0.015).

Flow Velocities (ft/s)			K Values			Equivalent Pipe Length (ft)		
Vm	Va	Vb	Km	Ka	Kb	Lm	La	Lb
Main Line with Two Perpendicular Laterals								
8.7	4.7	4.7	0.615	0.381	0.390	97.4	138.5	141.8
8.5	3.5	6.2	0.549	0.264	0.373	89.8	176.8	78.3
8.5	2.9	2.7	0.535	0.085	0.108	61.1	56.9	83.9
8.1	3.0	7.4	0.610	0.365	0.516	110.4	332.6	75.2
7.3	6.5	5.5	0.756	0.665	0.641	170.2	129.1	173.8
6.8	5.5	8.4	0.792	0.682	0.849	227.8	202.9	108.8
6.7	2.5	10.3	0.920	0.729	1.121	243.0	956.7	83.4
6.1	10.1	4.3	0.871	1.095	0.797	286.7	89.8	361.9
6.0	7.1	7.6	0.835	0.838	0.872	288.1	140.3	127.1
5.9	12.0	3.1	0.957	1.329	0.806	346.5	79.0	734.5
5.6	6.6	8.9	0.884	0.876	1.035	353.7	170.1	110.9
4.3	7.3	6.7	1.003	1.157	1.051	498.4	131.3	143.5
4.2	7.0	6.2	1.017	1.123	1.029	477.6	127.5	150.0
4.1	7.7	7.5	0.915	1.056	1.021	543.9	119.9	123.0
4.0	12.3	2.9	1.006	1.605	1.009	598.0	69.6	783.5
4.0	10.6	3.9	0.928	1.391	0.964	518.7	76.0	390.4
3.7	8.1	7.0	1.003	1.242	1.130	678.3	119.1	144.8
3.7	6.6	5.6	0.902	1.045	0.902	448.2	111.9	131.5
3.2	6.0	5.0	1.048	1.184	1.047	551.8	119.9	152.7
3.0	5.6	4.6	0.953	1.102	0.974	501.8	111.6	142.0
2.7	11.3	3.8	1.060	1.753	1.142	1120.1	70.8	415.1
2.6	11.1	9.6	0.904	1.300	1.180	1698.2	88.1	107.2
2.4	13.3	7.7	0.994	1.607	1.287	2093.4	78.0	187.7
2.4	4.2	4.2	0.805	0.983	1.008	423.8	118.5	121.5
2.2	5.3	4.0	0.948	1.200	1.045	641.1	98.4	152.4
Main Line with One Perpendicular Lateral								
8.5	6.5		0.607	0.408		75.4	59.5	
8.3	9.4		0.778	0.817		127.3	70.5	
6.9	11.8		0.961	1.279		224.9	69.9	
6.7	13.4		1.066	1.527		293.7	71.3	
5.3	14.7		1.087	1.792		435.0	63.2	
4.1	14.7		1.206	2.140		674.1	62.6	
2.2	14.6		1.227	2.617		1693.5	55.0	
Main Line Only								
12.7			0.095			5.8		
7.7			0.105			6.4		
6.0			0.020			1.2		
Two Perpendicular Laterals, no Main Line								
	12.8	3.0		2.594	2.829		51.9	1028.4
	10.1	5.9		2.212	2.127		73.1	203.9
	9.1	8.1		1.574	1.427		73.5	84.8

The actual pipe length remains unchanged. Figure 4 is a schematic of the actual pipeline and manhole system showing the relationship between actual configuration and equivalent pipe length in a numerical model. Lm and L_a (or L_b) represent the equivalent pipe length for the main and lateral, respectively.

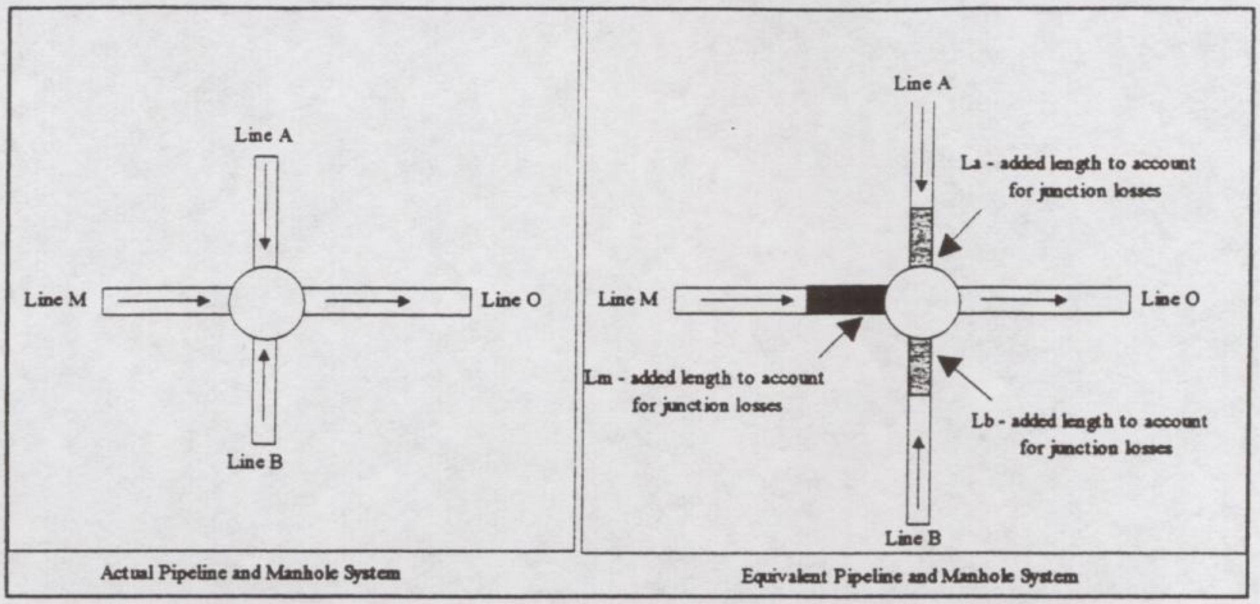

Figure 4. Equivalent Length Concept

The calculation of equivalent pipe length is based on the Darcy-Weisbach equation

$$h_L = f\frac{L}{D}\frac{V^2}{2g} \tag{4}$$

where h_L is the general term for energy loss due to friction, f is the friction coefficient, L is the pipe length, and D is the pipe diameter. The equation is combined with Chezy's formula and Manning's equation to relate the friction coefficient to Manning's roughness coefficient (British units system)

$$f = 5.75\frac{gn^2}{\sqrt[3]{D}} \tag{5}$$

The equivalent pipe length is then calculated using

$$K_m\frac{V_o^2}{2g} = 5.75\frac{gn^2}{\sqrt[3]{D}}\frac{L_m}{D}\frac{V_m^2}{2g} \tag{6}$$

and solving for Lm.

$$L_m = \frac{D^{\frac{4}{3}}}{5.75gn^2}\frac{V_o^2}{V_m^2}K_m \tag{7}$$

Writing a similar expression for the lateral and solving gives the following formula for L_a .

$$L_e = \frac{D^{\frac{4}{3}}}{5.75gn^2} \frac{V_o^2}{V_e^2} K_e \qquad (8)$$

The equivalent pipe lengths are shown in Table 1 for n=0.015. Calculating equivalent lengths for any n value is accomplished by simply changing the n value in the formulas and recalculating the equivalent lengths. One unexpected feature is that some head loss coefficients are negative. At first glance this result does not make sense, however the result is correct and energy is conserved in the system. The negative coefficients arise because of our choice of a traditional control volume where the inlet is selected just upstream of each junction entry. An alternative control volume approach is discussed in detail by Serre et al. (1994). When our data are analyzed using their approach, all the loss coefficients are positive. We elected to remain with the traditional approach to be consistent with the minor loss concepts of pressurized pipe flow and loss concepts of open channel flow, all of which use the traditional control volume inlet location.

Measurement Procedure

The head-loss coefficients are determined from measured values of piezometric and kinetic energy for different flow rates and different model configurations. The general measurement procedure (after daily calibration) is as follows:

- System control valves are set and the flow is allowed to reach a steady state (15-30 minutes)
- Outlet-line weir depth, Inlet flow velocities, and manometer heights are measured.
- Piezometric head H_p = manometer reading.
- Kinetic energy head $H_v = V^2/2g$
- Total head $H_t = H_p + H_v$ is computed for each manometer and plotted against manometer location using the model center as the reference origin.
- Best-fit lines are calculated for each pipe line, and extended through the model center.
- The difference in intercept heights is the total head loss due to the manhole.
- The ratio of head loss for a given pipe line to kinetic energy head in the outlet line is the head loss coefficient.

Figure 5 is a typical energy grade line plot that is used to develop the head loss coefficients for different flow ratios and configurations.

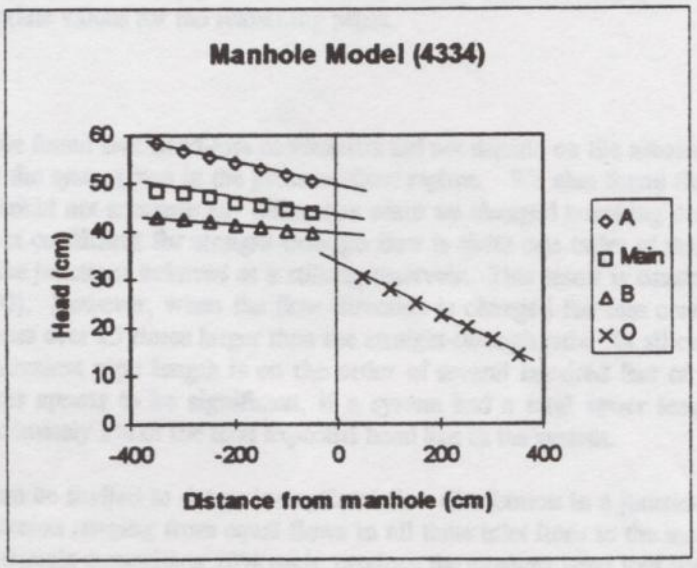

Figure 5. Typical Energy Grade Line Plot

Empirical Design Formulas

While the tabulated data is useful, it is tedious to interpolate in the table for flow configurations that were not actually measured. Furthermore, the equivalent pipe lengths change as the flow configuration changes, so a numerical model that is computing transient (dynamic) sewer water levels and flows cannot use a single equivalent pipe length value. To produce generalized predictive formulas all measurements for each configuration we studied were fitted by regression to the following equation form(s):

$$K_m = c_1X + c_2Y + c_3X^2 + c_4XY + c_5Y^2 + c_6X^3 + c_7X^2Y + c_8XY^2 + c_9Y^3 + c_0$$

$$K_a = d_1X + d_2Y + d_3X^2 + d_4XY + d_5Y^2 + d_6X^3 + d_7X^2Y + d_8XY^2 + d_9Y^3 + d_0$$

where $X=(q_a - q_b)/\sqrt{3}$, $Y=q_m$, and $q_i = \dfrac{Q_i}{Q_o}$ are flow fractions of the main and lateral lines with respect to the outlet (total) discharge. K_b is determined by symmetry, by interchanging the roles of Q_a and Q_b. Table 2 below lists the fitting coefficients for the K_m equation each configuration we studied.

Table 2a. Empirical Equation Coefficients for K_m Calculation

Configuration	c_1	c_2	c_3	c_4	c_5	c_6	c_7	c_8	c_9	c_o
4444	-0.027	-0.427	1.681	1.642	-1.073	-0.602	1.119	-1.535	-0.252	0.965
4334	0.000	0.699	0.926	0.000	-2.446	0.000	2.785	0.000	0.871	0.938
4224	0.000	-4.015	-1.513	0.000	7.938	0.000	13.630	0.000	-5.503	1.478

Table 2b. Empirical Equation Coefficients for K_a Calculation

Configuration	d_1	d_2	d_3	d_4	d_5	d_6	d_7	d_8	d_9	d_o
4444	0.000	-0.312	1.691	1.562	-1.418	-0.691	1.073	-1.442	0.000	0.959
4334	0.356	-2.147	7.656	4.747	0.884	-3.831	-6.909	-5.245	-1.299	1.718
4224	8.144	-8.407	30.910	-10.451	2.174	-33.219	-13.615	9.481	0.532	4.812

The configuration refers to the model pipe sizes used to develop the formulas. For example 4334 would be an "X" configured model, with a main inlet size of 4-inches, laterals A and B also 3-inches, and an outlet line of 4-inches. To compute values for "L" and "T" simply let the lateral flows vanish. For example 4X34 would be calculated using $q_a = 0$, and appropriate values for the remaining pipes.

Results and Conclusions

During the experiments we found that head-loss coefficients did not depend on the amount of surcharge as long as there was enough so that the system was in the pressure-flow regime. We also found that the base configuration also had little effect, we could not measure any difference when we changed benching details. The results of this study indicate that the loss coefficient for straight-through flow is about one order of magnitude smaller that one would predict assuming the junctions behaved as a stilling reservoir. This result is consistent with the findings of Pederson and Mark (1990). However, when the flow direction is changed the loss coefficient increases at least five-fold, and, in some cases over 25 times larger than the straight-through case. In all cases where flow direction is changed the added equivalent pipe length is on the order of several hundred feet of pipe. For even a small system, the junction losses appear to be significant, if a system had a total sewer length of 30,000 feet, each junction represents approximately 1% of the total expected head loss in the system.

The empirical formulas can be studied to determine optimal flow distribution in a junction. In the case of an "X" configuration, flow distribution ranging from equal flows in all three inlet lines to the main inlet comprising 60% of total flow and the two laterals comprising 20% each produce the smallest head loss coefficients. As the lateral flows become more unequal the lateral loss coefficients increase dramatically.

The empirical formulas are relatively simple, and when converted to equivalent pipe-length should allow engineers to easily incorporate variable junction losses into models that do not directly include junction losses. Further experiments are in progress to complete the empirical equation coefficients table and to include the effects of a 45-degree ("J") configuration.

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