JOURNAL OF THE AMERICAN WATER RESOURCES ASSOCIATION
VOL. 34, NO. 6 AMERICAN WATER RESOURCES ASSOCIATION DECEMBER 1998

HEAD LOSS AT MANHOLES IN SURCHARGED SEWER SYSTEMS[1]

K. H. Wang, T. G. Cleveland, C. Towsley, and D. Umrigar[2]

ABSTRACT: A laboratory investigation was conducted to determine the head losses at sewer pipe junctions (manholes) under surcharged conditions. A physical model of a manhole/pipeline system was constructed for head-loss measurements. Head-loss coefficients were determined for a variety of outlet-flow Reynolds numbers, surcharge levels, pipe sizes, flow configurations, and inlet-flow rates. Empirical formulas were also developed to estimate head-loss coefficients. The results indicate that head loss is insensitive to the amount of surcharge, but depends heavily on the flow configuration, relative flow rate, and the change of pipe diameter within the pipelines.
(KEY TERMS: hydraulics; physical modeling; manholes; head-loss coefficient.)

INTRODUCTION

Municipal sewerage systems are usually designed to operate without surcharging. That is, in each line, the water level is below the crown elevation of the conduit. Flow in the network is then gravity-driven, and is treated as open-channel flow. However, if the inflow exceeds the pipe-full capacity of the pipeline, or if it is affected by a backwater from a downstream flow constraint, the system will surcharge (pressurized flow). The head loss through a gravity flow junction is often neglected during normal flow conditions. However, when the system is surcharged manhole junction losses become important and can comprise a significant percentage of the overall losses within a sewer system. This is especially true in large systems with many junctions. Thus, it is essential to incorporate the effect of manhole head losses into the design of sewer pipe lines so that the system can store excess flow without flooding and overflows.

Although the junction head losses are important to the design of pipeline systems, the determination of head losses in surcharged manholes has not been studied extensively. Head losses in straight through and 45° bend manhole junctions were investigated by Acker (1959) and later by Hare (1983). The measurements of manhole head losses for a system with a 90° bend or a "T" junction were conducted by Marsalek (1985). He also examined the effect of different manhole benchings and found that the manhole head losses could be reduced by as much as 46 percent when flow guidance is provided at the base of a manhole junction. A submerged jet theory for the flow in straight through manholes was presented by Pedersen and Mark (1990) to determine the head losses in manholes.

Losses at sewer junctions depend on flow rate, junction geometry, and the change in pipe diameter between the inflow and outflow lines. In this study, we conducted a series of laboratory tests to determine the head-loss coefficients through a manhole under a variety of pipe configurations and flow rates. These head-loss coefficients can be used in predictive hydraulic models for the design and operation of sewer pipe lines.

MANHOLE HEAD LOSS AND HEAD-LOSS COEFFICIENTS

Under surcharged conditions, flow in each of the lines connected to a manhole is pressure-driven. At any point along these lines, the total head, H, consists

[1]Paper No. 97050 of the *Journal of the American Water Resources Association*. **Discussions are open until August 1, 1999.**
[2]Respectively, Associate Professors, Department of Civil and Environmental Engineering, University of Houston, Houston, Texas 77204-4791; Graduate Assistant, Department of Civil and Environmental Engineering, University of Houston, Houston, Texas 77204-4791; and Engineer, City of Houston, 1100 Louisiana, Suite 1100, Houston, Texas 77002 (E-Mail/Wang: khwang@uh.edu).

of a piezometric head and a velocity head. To describe the energy losses at a manhole junction, we apply the concepts of mass and energy balance. The control volume consists of the manhole itself along with three inlet sections and one outlet section. The energy loss per unit time ($\Delta\dot{E}$) is given by the energy balance equation,

$$\frac{\Delta\dot{E}}{\rho g} = Q_m\left(h_m + \frac{v_m^2}{2g}\right) + Q_a\left(h_a + \frac{v_a^2}{2g}\right) + Q_b\left(h_b + \frac{v_b^2}{2g}\right) - Q_o\left(h_o + \frac{v_o^2}{2g}\right) \quad (1)$$

where h_i = piezometric head in line i (i = m, a, b or o); v_i = velocity; Q_i = discharge; ρ = fluid density; g = acceleration due to gravity; and subscripts m, a, b, and o refer to the main, lateral A, lateral B and outlet lines, respectively (see Figure 1). The sum of pressure head, p/γ, and elevation head, z, is known as the piezometric head. The continuity equation gives $Q_o = Q_m + Q_a + Q_b$. The manhole energy loss described in Equation (1) essentially consists of loss from the main line and loss associated with two lateral inflows, which yields

$$\frac{\Delta\dot{E}}{\rho g} = Q_m \ \Delta H_m + Q_a \ \Delta H_a + Q_b \ \Delta H_b \quad (2)$$

where ΔH_m = manhole head losses corresponding to the main inlet line; and ΔH_a and ΔH_b = manhole head losses corresponding to lateral line A and lateral line B, respectively. From Equations (1) and (2) and the continuity equation, we have

$$\Delta H_m = \left(h_m + \frac{V_m^2}{2g}\right) - \left(h_o + \frac{V_0^2}{2g}\right) \quad (3)$$

$$\Delta H_a = \left(h_a + \frac{V_a^2}{2g}\right) - \left(h_o + \frac{V_0^2}{2g}\right) \quad (4)$$

and

$$\Delta H_b = \left(h_b + \frac{V_b^2}{2g}\right) - \left(h_o + \frac{V_0^2}{2g}\right) \quad (5)$$

The head loss term ΔH_i (i = m, a, or b) is traditionally expressed as the product of a dimensionless head-loss coefficient, K_i (i = m, a, or b), and the outlet velocity head. The dimensionless head-loss coefficient, K_i, can then be determined as

$$K_i = \Delta H_i \,/\, \frac{V_o^2}{2g} \qquad (i = m, a, or\, b) \quad (6)$$

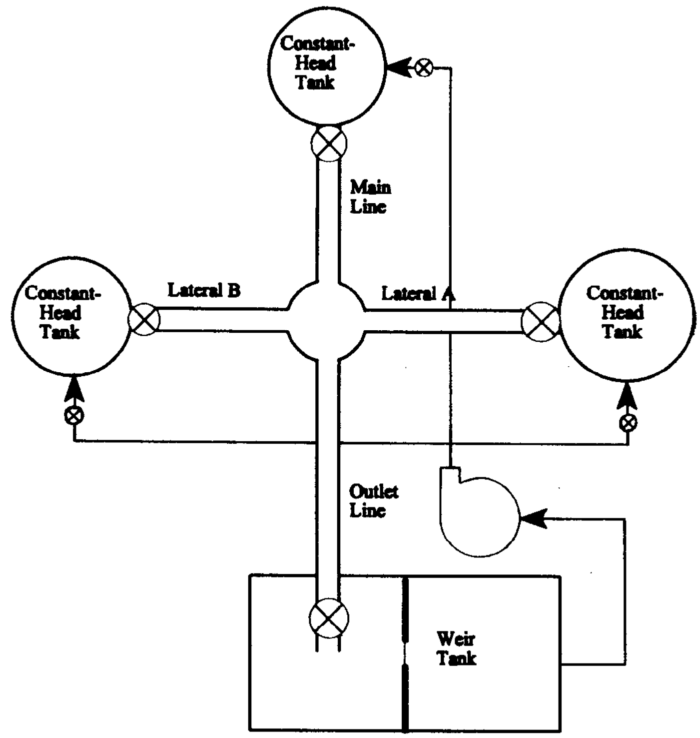

Figure 1. Schematic Layout of the Physical Model.

VARIABLES AFFECTING HEAD-LOSS COEFFICIENTS

Figure 2 is a schematic diagram showing the geometrical variables which may affect the performance of a typical manhole junction. In general, the head-loss coefficient may be assumed to be a function of the fluid density, ρ; the dynamic viscosity, μ; the gravitational constant, g; the discharges in the main, lateral A, lateral B, and outlet lines, Q_m, Q_a, Q_b and Q_o (note that by conservation of mass, only three of the flow rates are independent); the surcharge depth in the manhole, S; the manhole base diameter, D; and the inlet and outlet line diameters, d_m, d_a, d_b and d_o. In the present study, only one type of manhole benching was used, so its effect on the head-loss coefficient is ignored. We thus have identified twelve variables which may affect the head-loss coefficient. By the

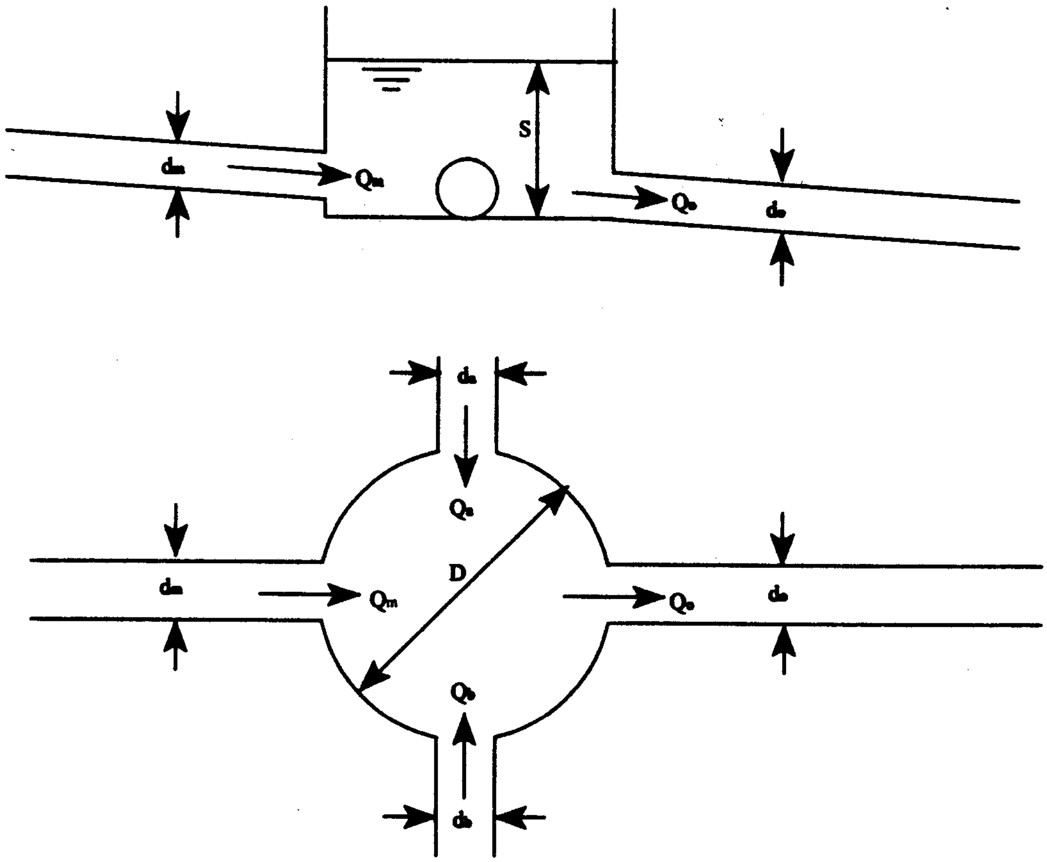

Figure 2. Junction Geometrical Parameters.

Buckingham π Theorem, we can define nine dimensionless variables to describe each head-loss coefficient. They can be expressed as

$$\frac{Q_o}{g^{1/2}d_o^{5/2}}, \frac{\rho Q_o}{\mu d_o}, \frac{S}{d_o}, \frac{Q_m}{Q_o}, \frac{Q_a}{Q_o}, \frac{D}{d_o}, \frac{d_m}{d_o}, \frac{d_a}{d_o}, \frac{d_b}{d_o}$$

Since Q_o is equal to $v_o \pi d_o^2/4$, the first two variables are just the Froude Number and the Reynolds Number. Among the independent variables, the first five are flow characteristics and the last four describe the junction geometry. In surcharged flow, the head-loss coefficient does not depend on the Froude Number. The effects of Reynolds Number, the surcharge depth, the relative inflows and the pipe diameters on the manhole head losses were examined in this study.

MODEL CONSTRUCTION AND TEST PROCEDURE

A 1:6 scale physical model was constructed in the Hydraulics Laboratory at the University of Houston. The model layout is illustrated in Figure 1. The physical modeling system was about 40 feet long in the main line (from head tank to weir tank) and 20 feet long in each lateral line. The manhole junction was connected to a main inlet line, designated "M," and two lateral lines, designated "A" and "B," flowing into the model, and a single outlet line, designated "O," leaving the model. The pipes were supported by eight steel-frame tables which could be adjusted to any slope by means of screw jacks. A 1/1000 pipe slope was used for the test. A 5 hp pump was used to supply water from storage tanks to three head tanks to provide the head needed for circulating the flow through the system.

The manhole model built for the test was an 8 inch diameter and 40 inch high circular manhole base with four openings around its circumference at the base. Semi-circular flow channels were cut into the base to model the typical "benching" of manhole design (Figure 3). By blocking one or more of the inlets with plugs and using pipe adapters, a variety of flow configurations with different pipe diameters (2 inch, 3 inch, and 4 inch) were investigated.

Each line was equipped with six manometers. Readings from these manometers established a

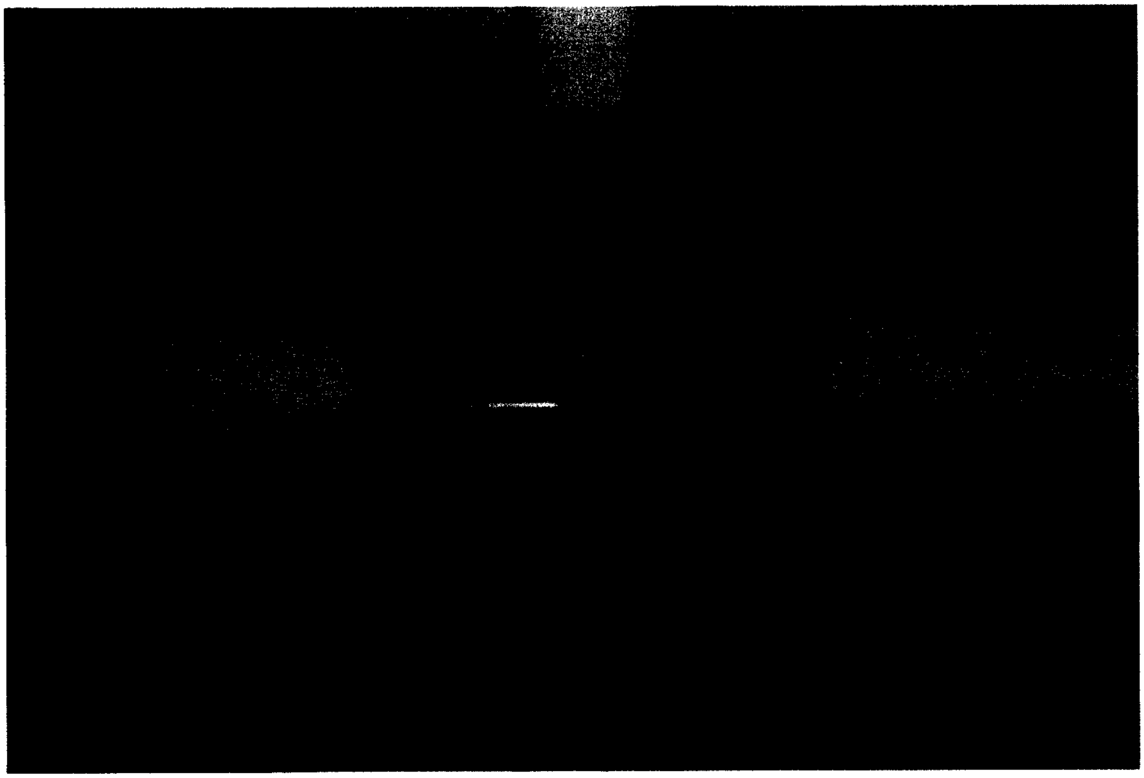

Figure 3. Inside of Manhole Base (outlet on the right) (benched for 4-inch main line and 3-inch laterals).

hydraulic grade line (a line representing the variation of hydraulic head which is equal to elevation head plus pressure head) in each inlet line and outlet line. An ultrasonic Doppler-shift flowmeter was attached to each line to measure the velocity in that line, from which the velocity head, $v^2/2g$, was calculated. The outflow discharge, Q_o, was measured at the outlet using a calibrated sharp-crested weir. The ultrasonic flowmeter for each line was calibrated by comparison to the weir discharge measurement. Control valves at the inlet and outlet of each constant-head tank could be adjusted to establish the desired flow condition. Surcharge level could be controlled with the valve at the downstream weir tank. When the flow stabilized, measurements of weir head, inlet line velocities, and manometer heights were recorded.

Tests were conducted for various flow rates, pipe sizes and for flow configurations of straight, T junction, cross, and 90° bend. Flow configuration is indicated by a four-number notation $d_m d_a d_b d_o$ with inlet and outlet line diameters d_m, d_a, d_b, and d_o in inches; for example, 4334 indicates that the main line diameter is 4 inches, lateral lines A and B are both 3 inches, and the outlet line diameter is 4 inches. An X in the configuration notation indicates that the corresponding line is plugged, for example, X334 indicates a T-junction with 3-inch laterals and a 4-inch outlet line. To simplify the notation of dimensionless flow rate, we define $q_m = Q_m/Q_o$, $q_a = Q_a/Q_o$, and $q_b = Q_b/Q_o$.

DETERMINATION OF HEAD-LOSS COEFFICIENTS

The recorded data can be used to determine the manhole head-loss coefficients for the main inlet line and the lateral lines. The total energy head (equal to the piezometric head plus the velocity head) is computed for each manometer and plotted against manometer distance from the center of the model. The best-fit straight lines shown in Figure 4 display the total energy grade lines for the main inlet line, lateral A, lateral B and the outlet line respectively. The best-fit straight lines are extended through the model center. The difference in intercepts between each inlet line and outlet line represents the junction head loss induced by that inlet line at the manhole junction. The head-loss coefficient for each inlet line is then determined using Equation (6).

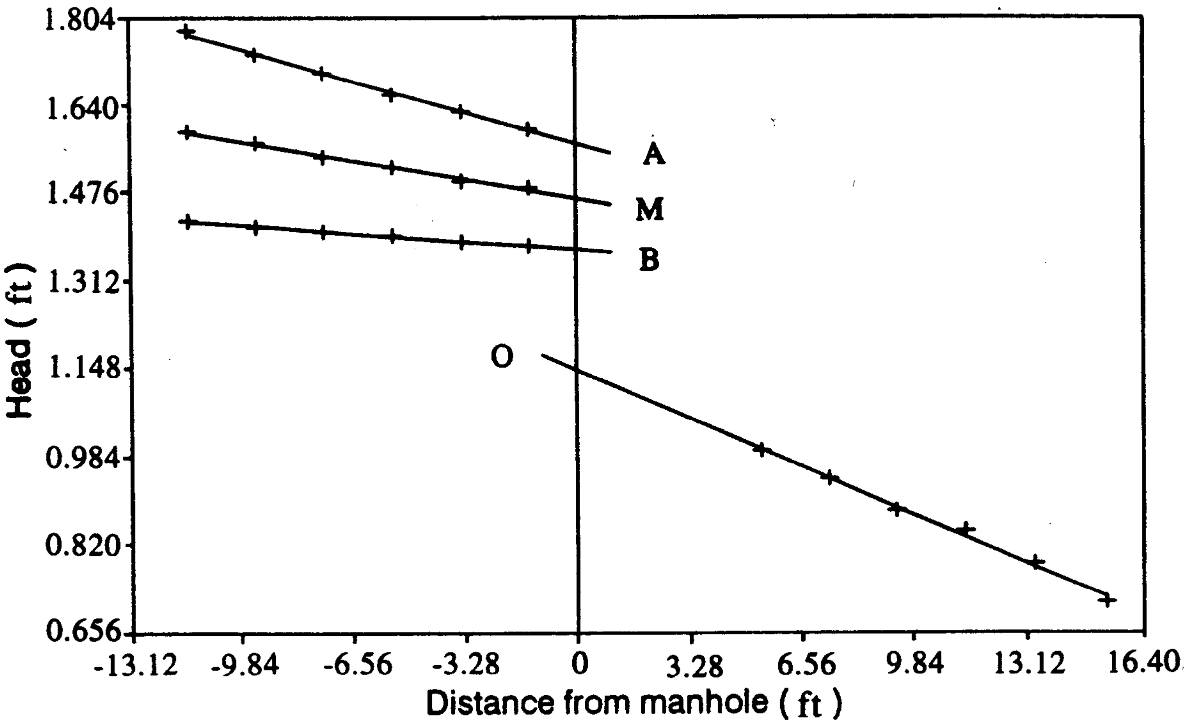

Figure 4. A Typical Energy Grade Line Plot to Show the Head Losses Between the Inlet Lines and the Outlet Line.

RESULTS AND DISCUSSION

The manhole head losses induced by each inlet line were measured for all flow configurations and mixed pipe sizes. For each test case, the data were analyzed and the head-loss coefficients were calculated. The dependence of the head-loss coefficients on outflow Reynolds number and surcharge level are examined first. The head-loss coefficient K_a for inlet pipe A is plotted versus the outflow Reynolds number (Re) for the cases of 4334 and 4444 configurations in Figures 5 and 6, respectively. The results indicate that the head-loss coefficients are independent of Reynolds number, at least for Re values greater than 10^4. This conclusion is consistent with the results presented by Marsalek (1985). The dependence of head-loss coefficients on surcharge level was also studied. The measured data reveal that the head-loss coefficients are also independent of surcharge level. (The results are not shown here, but can be found in Wang *et al.*, 1995.)

A series of model tests was conducted to determine the head-loss coefficients for each inlet line under various flow configurations, flow rates, and pipe diameters (e.g., 4444, 4334, 4224, 3224, and 3334 configurations). A complete summary of the measured data and calculated head-loss coefficients for all test cases can be found in Wang *et al.* (1995). Considering the effect of inlet flow rate, a set of empirical formulas for determining the head-loss coefficients K_m and K_a were developed using a regression equation approach. They are

$$K_m = M_0 + M_1Y + M_2X^2 + M_3Y^2 + M_4X^2Y + M_5Y^3 \quad (7)$$

$$K_a = A_0 + A_1X + A_2Y + A_3X^2 + A_4XY + A_5Y^2 + A_6X^3 + A_7X^2Y + A_8XY^2 + A_9Y^3 \quad (8)$$

where $X = \frac{q_a - q_b}{\sqrt{3}}$, $Y = q_m$, and q_m, q_a, and q_b are flow fractions of the main inlet line, and lateral lines A and B versus the outlet flow rate. The coefficients M_0 to M_5 and A_0 to A_9 for the flow configuration of 4444, 4334, 4224, 3224, and 3334 are summarized in Table 1 and Table 2, respectively. The correlation coefficient, R^2, is also provided in Table 1 and Table 2 to show the degree of agreement between the measured head-loss coefficients and the fitted polynomial. A third-order polynomial was found to give a consistent and satisfactory correlation coefficient ($R^2 \approx 1$) for all test cases. Error analysis for the predicted head-loss coefficient was also conducted by calculating the root mean squared error. The error range for

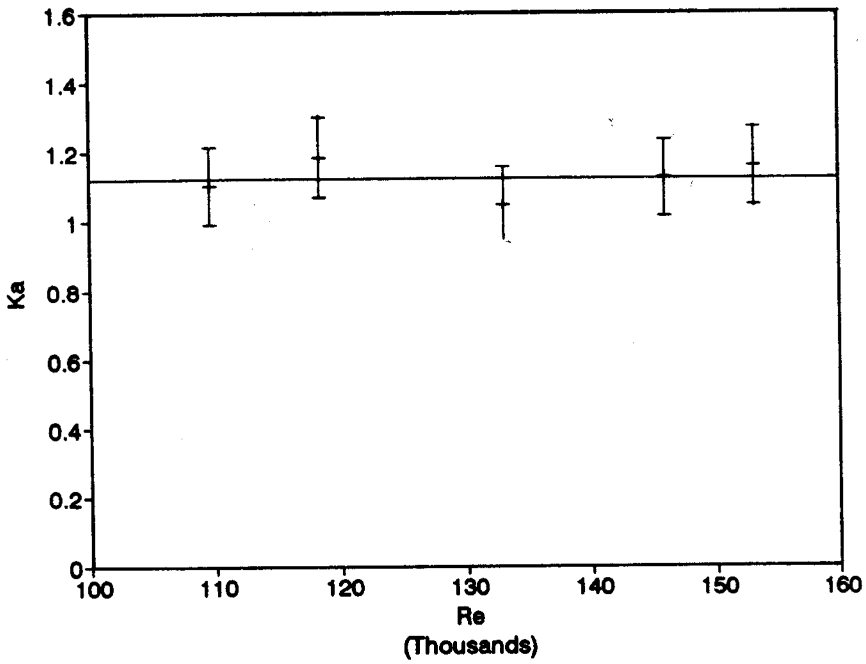

Figure 5. Values of K_a Measured for Various Outflow Reynolds Numbers ($q_m \approx 35$ percent, $q_a \approx 35$ percent, $q_b \approx 30$ percent in 4334 configuration).

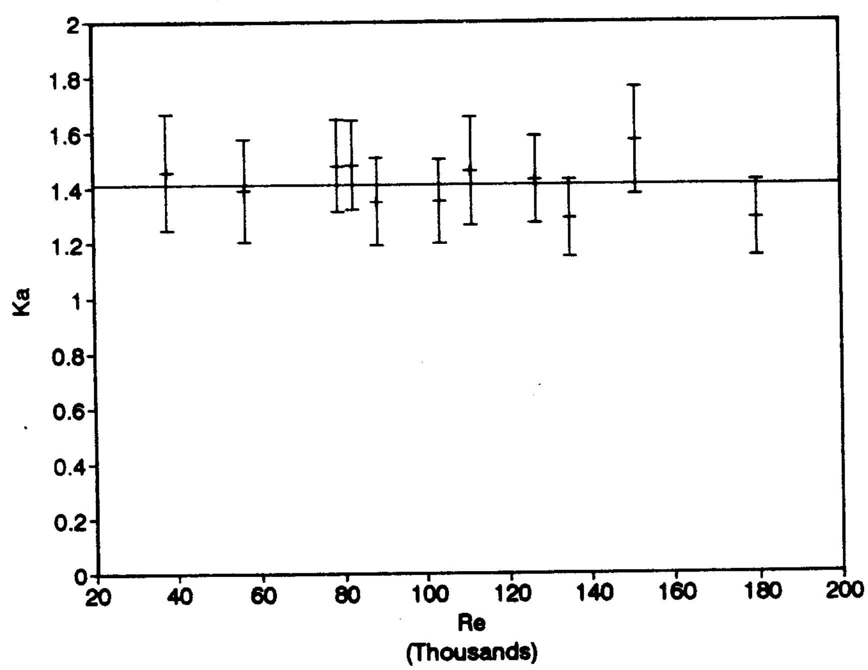

Figure 6. Values of K_a Measured for Various Outflow Reynolds Numbers (90° bend in a 4-inch pipeline or X4X4 configuration).

each test configuration is given in Table 1 and Table 2. To calculate K_b, use the K_a formula with q_a and q_b interchanged.

To transfer the empirical formulas of the head-loss coefficients into graphical use, triangular contour plots of K_m and K_a can be constructed. Here, we present the contour plots of K_m and K_a for the pipeline configuration of a 4-inch main line and 2-inch laterals in Figures 7(a) and 7(b), respectively, to illustrate the use of the triangular contour plots and to describe the

TABLE 1. Coefficients of the Empirical Formula in Equation (8) for Determining K_m.

Configuration	M_0	M_1	M_2	M_3	M_4	M_5
4444 (R^2 = 0.88) Error = ± 0.077	0.74	0.65	0.01	-2.38	3.92	1.08
4334 (R^2 = 0.93) Error = ± 0.074	0.94	0.7	0.93	-2.45	2.78	0.87
4224 (R^2 = 0.91) Error = ± 0.140	1.46	-2.67	0.72	4.21	9.13	-3.03
3224 (R^2 = 0.93) Error = ± 0.095	1.23	-1.84	1.44	5.11	9.65	-3.87
3334 (R^2 = 0.73) Error = ± 0.086	0.82	-0.72	0.86	3.92	0.47	-3.26

TABLE 2. Coefficients of the Empirical Formula in Equation (9) for Determining K_a.

Configuration	A_0	A_1	A_2	A_3	A_4	A_5	A_6	A_7	A_8	A_9
4444 (R^2 = 0.97) Error = ± 0.09	0.97	-0.03	-0.43	1.68	1.64	-1.07	-0.60	1.12	-1.54	-0.25
4334 (R^2 = 0.99) Error = ± 0.10	1.72	0.36	-2.15	7.66	4.75	0.88	-3.83	-6.91	-5.43	-1.30
4224 (R^2 = 0.99) Error = ± 0.39	4.82	7.00	-8.88	37.97	-3.76	3.79	-39.04	-29.28	2.30	-0.63
3224 (R^2 = 0.99) Error = ± 0.52	4.80	-3.80	-10.54	30.53	20.42	8.33	3.22	-20.55	-14.21	-4.76
3334 (R^2 = 1.0) Error = ± 0.12	1.61	0.54	-3.14	6.91	5.37	3.93	-2.88	-7.46	-7.07	-4.77

corresponding variations of the head-loss coefficients. In Figure 7, each of the q_i (i = m, a, or b) is scaled linearly from the side of the triangle labeled "$q_i = 0$" to the opposite vertex, labeled "q_i = 100 percent." At any point inside the triangle, $q_m + q_a + q_b$ = 100 percent. The contour lines show the corresponding K_m and K_a values. It is found that the head-loss coefficients are strongly dependent on the relative flow rates in the pipeline and on the flow configurations (e.g., straight through, T, cross, and 90° bend). The results indicate that the head loss is relatively insignificant for a straight through configuration ($q_a = 0$ and $q_b = 0$). However, the head loss is not negligible in the manhole junctions when there exists lateral inflows ($q_a \neq 0$ or $q_b \neq 0$), or when the junction forces a change in flow direction. Also, as the lateral inflows (flows from line A and line B) become more unequally distributed, the head-loss coefficients for the lateral lines increase dramatically. The triangular contour plots of the head-loss coefficients for other types of flow configurations can be found in Wang *et al.* (1995).

The variations of K_m and K_a versus flow rate of q_a for the 44X4 (T junction) configuration are plotted in Figures 8 and 9, respectively. The present data are

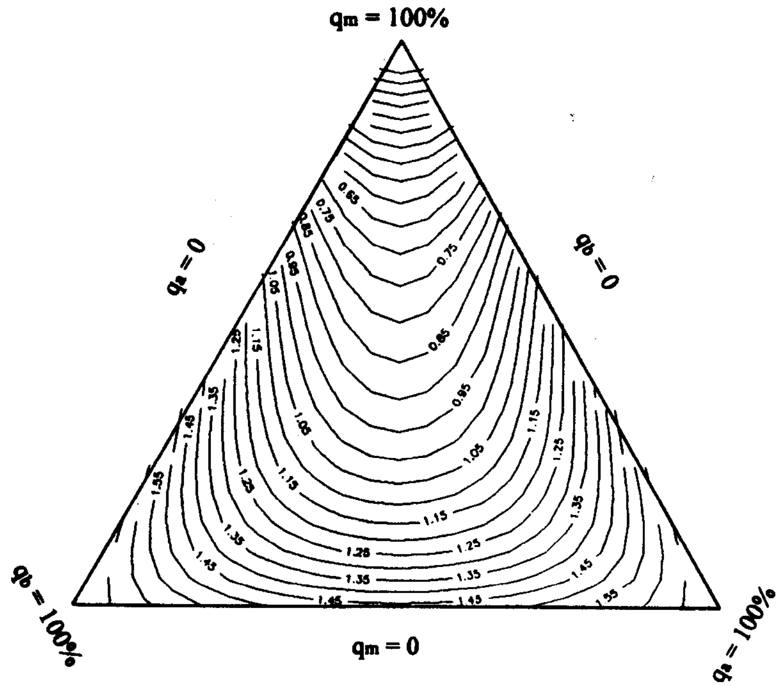

Figure 7(a). K_m Contour Plot for the Flow Configuration of 4224.

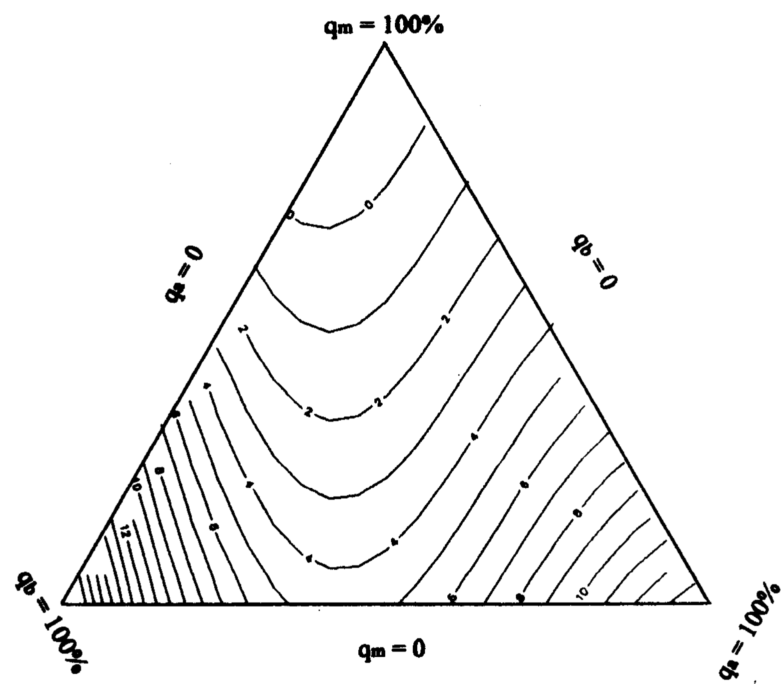

Figure 7(b). K_a Contour Plot for the Flow Configuration of 4224.

marked by open squares. The results of Marsalek (1985) using an equivalent flow configuration and manhole model (Mould M2 – a semi-circular flow channel is cut into the base of the manhole model) are also presented as filled triangles for comparison. The error bars corresponding to the present measurements are provided in Figures 8 and 9 to indicate the range of error that occurred during laboratory tests. However, Marsalek (1985) did not provide error information in his manuscript. Therefore, no error bar is

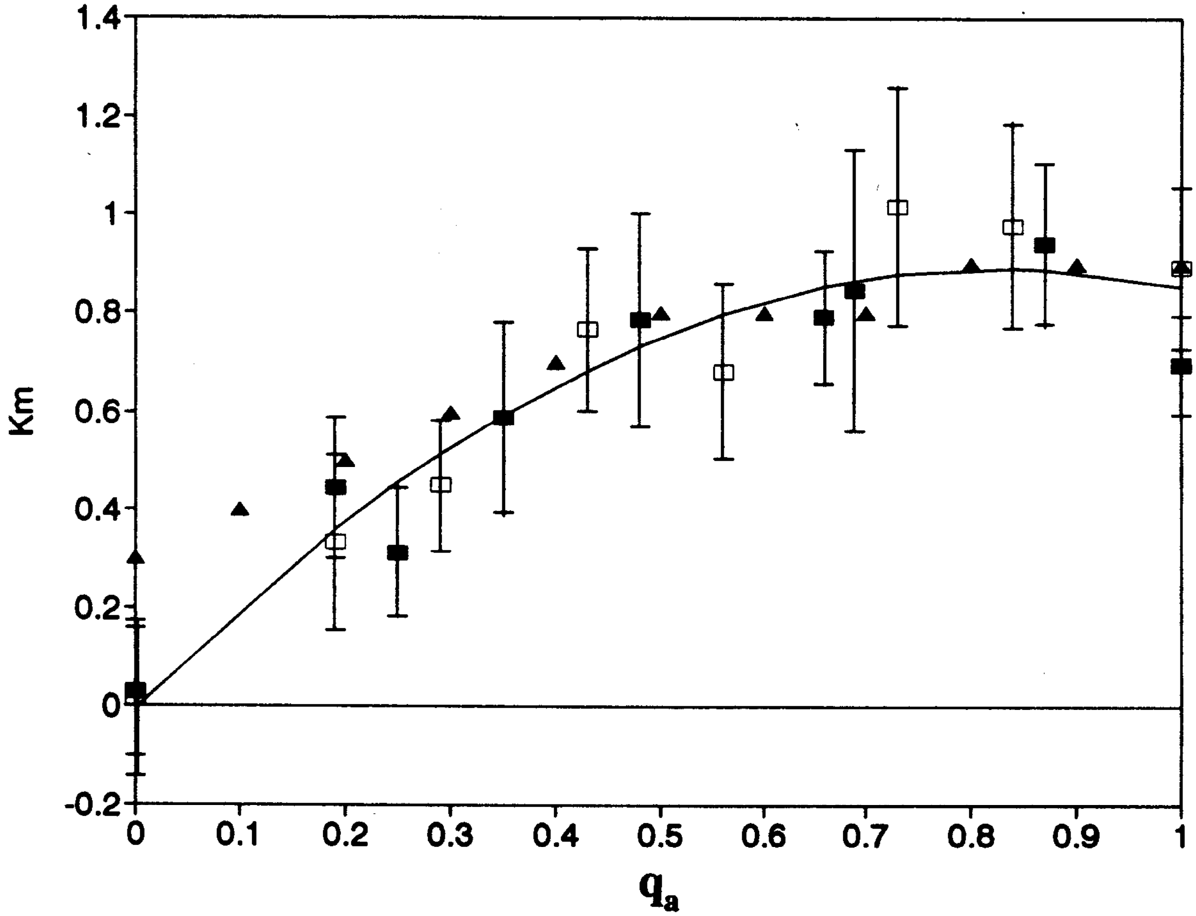

Figure 8. Variation of K_m Versus Flow Rate of q_a for (a) 44X4 (T) Junction: Open Squares; (b) 4444 with $q_b = 0$: filled squares; (c) Marsalek (1985): Filled Triangles; and (d) Solid Line: Best Fitted Line.

plotted with Marsalek's results. The agreement between present measurements and Marsalek's is generally good. In Figures 8 and 9, the head-loss coefficients for 4444 with $q_b = 0$ are presented as filled squares. Within the estimated errors, it is interesting to note that the head-loss coefficients for both configurations (44X4 with line B plugged and 4444 with $q_b = 0$) are nearly the same, indicating that a no-flow line can be considered as a line which is hydraulically disconnected from the system.

CONCLUSIONS

In this study, we built a physical model to measure the manhole head losses. Empirical formulas to estimate the head-loss coefficients were also developed. Our studies found that the head-loss coefficients are strongly dependent on the relative flow rate and the change of pipe diameter within the pipelines. We observed that the junction head loss is negligible for a straight through configuration. However, the head losses become significant in the manhole junctions when there exists lateral inflows or the junction forces a change in flow direction. In some of these cases, head-loss coefficients can be more than 25 times larger than the straight-through case. We also found that as the lateral flows become more unequally distributed, the lateral loss coefficients increase dramatically.

ACKNOWLEDGMENTS

This study was sponsored by the Greater Houston Wastewater Program, City of Houston under Contract 34125.

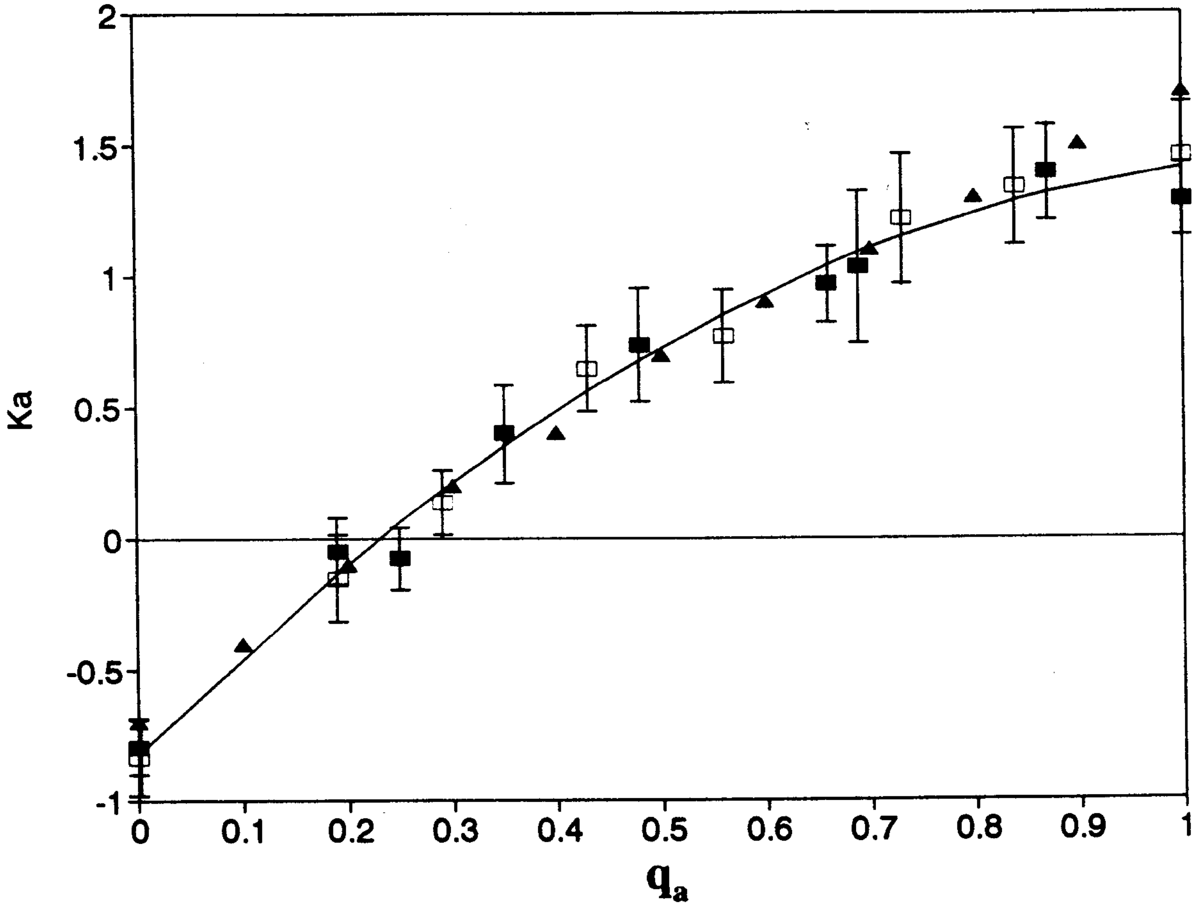

Figure 9. Variation of K_a Versus Flow Rate of q_a for (a) 44X4 (T) Junction: Open Squares; (b) 4444 with $q_b = 0$: Filled Squares; (c) Marsalek (1985): Filled Triangles; and (d) Solid Line: Best Fitted Line.

LITERATURE CITED

Acker, P., 1959. An Investigation of Head Losses at Sewer Manholes. Civil Engineering and Public Works Review 54:882-884.

Hare, C. M., 1983. Magnitude of Hydraulic Losses at Junctions in Piped Drainage System. Civil Engineering Transactions, The Institute of Engineers, Australia, pp. 71-76.

Marsalek, J., 1985. Head Losses at Selected Sewer Manholes. Environmental Hydraulics Section, Hydraulics Division, National Water Research Institute, Canada Center for Inland Waters.

Pedersen, F. B. and O. Mark, 1990. Head Losses in Stormsewer Manholes: Submerged Jet Theory. Journal of Hydraulic Engineering, ASCE 116(11):1317-1328.

Wang, K. H., T. G. Cleveland, and C. W. Towsley, 1995. Physical Modeling to Determine Head Loss at Selected Surcharged Sewer Manholes. Final Report Submitted to Greater Houston Wastewater Program, City of Houston.