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Minor Loss Coefficients for Storm Drain Modeling with SWMM

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In the 1990s, as one of the updates to version 4.0 of SWMM, an EXTRAN routine was included which allowed the modeler to include input parameters on the C1 card for calculating minor losses in flow transitions. The ability to model these types of losses explicitly, and to track them as part of the input file, is an important addition to EXTRAN's capabilities. Particularly in cases where there is no flow data for the storm drain network being modeled, it is important to try to represent all the physical processes that can have an effect on flows and water quality.

The goal of this chapter is to review commonly used loss coefficients to determine their applicability for modeling storm drains in EXTRAN. The literature review found that many of the coefficients in use were derived from experimental data unrelated to the type of flows found in storm drains. There was.much more information for pressure flow scenarios than for free surface flow. There was also more information for transitions in pipes than for transitions through junctions such as manholes.

In reviewing the original experimental research used to derive these coefficients, the chapter identifies several methods of estimating them which should be more applicable to storm drain modeling. The chapter also recommends that researchers conduct new experimental studies with the goal of developing loss coefficients that can be used for better estimates of junction losses in pipes with free surface flow.

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23.1 Introduction

In the 1990s, as one of the updates to version 4.0 of SWMM, an EXTRAN routine was included which allowed the modeler to include input parameters on the C1 card for calculating minor losses in flow transitions. The variables are KENTR, KEXIT, and KOTHER, which represent the coefficients for entrance, exit, and other losses (e.g. expansion, contraction, bends, or valves). The minor loss equation in EXTRAN takes the form of:

$$h_L = K\frac{v^2}{2g} \tag{23.1}$$

where:

h_L = the head loss
K = a loss coefficient
v = the velocity in the conduit, and
g = the gravitational constant

The ability to model these types of losses explicitly, and to track them as part of the input file, is an important addition to EXTRAN's capabilities. Particularly in cases where there is no flow data for the storm drain network being modeled, it is important to try to represent all the physical processes that can have an effect on flows and water quality. Minor losses (head losses from flows through junctions and bends) can be significant. UWRRC (1992) said that

> In long conduits where L/D >> 1000 these local losses are usually very small in comparison to the friction losses and…can be neglected. However, if the channel or conduit is very short and/or there are a number of manholes, changes in direction, junctions, or changes in pipe size, then the sum of these losses can exceed the friction loss.

23.1.1 Objective

The goal of this chapter is to develop loss coefficients for use in EXTRAN to model minor losses for storm drain design and analysis based on original experimental research. The method of finding loss coefficients should be simple to use, and where simplifications are made from the research, they should result in only minor variations from the more accurate calculations.

23.1.2 Modeling versus Design

There are differences in the process of head loss calculations between modeling and design. First, the modeler may not take into account every pipe and junction, but instead folds several short pipes into one longer equivalent pipe to improve model stability. A designer, on the other hand, must calculate the HGL for every part of the network to be built. Similarly, a SWMM model typically will ignore inlets and small branches, lumping them into RUNOFF calculations, while inlet and branches are a key part of storm drain design.

A second difference is in the type of flow. Designers typically work with a single peak flow value and design a closed storm drain system for full flow conditions. In a modeling situation, particularly with continuous models, a wide range of flows may occur. A conduit may operate in open channel, full flow, or pressure flow condition. Unlike the design situation, it is not possible for a modeler to predict in advance what flow regime will occur. For this reason, the head loss equation and coefficients should take into account the flow regime.

23.1.3 Conditions to be Modeled

Table 23.1 shows the conditions to be modeled, described in the EXTRAN manual:

Table 23.1 Minor losses modeled in Extran.

Variable	Condition	Description
KENTR	Entrance	flows entering a conduit from a larger waterbody
KEXIT	Exit	flows exiting a conduit to a larger waterbody
KOTHER	Expansion	transition from a smaller to larger conduit
	Contraction	transition from a larger to smaller conduit
	Bends	change in flow direction
	Valves	gate valve or other flow control device
	Inlet	flows entering the conduit from an inlet

Some of these losses are unlikely to be included in a model. Individual inlets and their losses, for example, are frequently assumed in the RUNOFF block and not modeled explicitly. Some may only be modeled occasionally.

Most minor losses in a closed storm drain system will take place in a junction, usually a manhole structure, where the flow enters through one or more pipes and exits through a single conduit. This situation is more complex than either the entrance and exit situation where flow is entering or leaving a pipe network or a simple expansion or contraction in pipe size.

The key to modeling losses in a storm drain network is understanding and developing computations for losses in junctions. These, in turn, will need to be modified so that they can be entered into EXTRAN as entrance, exit, and other losses for each conduit.

While there is some research on junction losses, most has been done for pressure flow situations. There has been much less study done on the conditions of flow in pipes flowing partially full, under free surface conditions. As shown in Table 23.2, these are the situations that are most frequently modeled for storm drain analysis.

Table 23.2 Types of minor losses to be found in storm drain models.

Type of Loss	Frequently Modeled	Occasionally Modeled	Rarely Modeled
Pipes (Full or Partially Full)			
Entrance		X	
Exit		X	
Expansion and Contraction		X	
Inlet on branch			X
Curves or bends		X	
Outfall		X	
Junctions (Full or Partially Full)			
Flow through junction	X		
Bend within junction	X		
Junction with lateral	X		
Junction with inlet			X
Channels			
Expansion and Contractions		X	
Curves or bends	X	X	
Culvert entrance	X		
Culvert exit	X		
Outfall		X	

23.2 Literature Review

The literature review had two goals. The first was to review readily available design criteria, textbooks, handbooks, and manuals to identify commonly accepted engineering practice. These included the following list. Journal articles were reviewed to identify methods and criteria for calculating minor losses, but these were not found to be in general use in engineering practice.

Design Criteria:
- Highway Drainage Manual (MSHA 1981)
- Virginia Drainage Manual (VDOT 1988)
- Urban Storm Drainage Criteria Manual (UDFCD 1969)

Handbooks
- Handbook of Hydraulics (King and Brater, 1963)
- Standard Handbook for Civil Engineers (Merritt, 1983)
- Design of Small Dams (USBR, 1977)
- Modern Sewer Design (AISI, 1980)
- Design and construction of urban stormwater management systems (UWRRC 1992)
- Handbook of Applied Hydraulics (Davis, 1954)

Software Manuals
- HEC-RAS
- HYDRAIN
- HEC-22

Textbooks
- Chow, 1959
- Henderson, 1966
- French, 1985
- Daugherty, Franzini, and Finnemore, 1985
- Streeter and Wylie, 1975
- Linsley and Franzini, 1975

Several problems were found with the methods and data used in the design criteria manuals, software manuals, and handbooks listed above:

1. There are few commonly accepted values or calculation methods. No two sources approached every situation in the same way.
2. Within a single reference, there may be two or more methods presented to calculate one type of loss. Frequently these methods do not give the same result.
3. Many of the design criteria do not have explicitly sourced ties to research verifying the data, so there is no way to check its validity.
4. Some of the criteria and data which can be tied to original research have been extrapolated to design situations that may be inappropriate.
5. While there is research and a commonly used standard for pressure flow in pipes and junctions, culvert entrance and exit losses, and open channel flow in channels, there remains almost no easily used research results on losses in junctions.
6. There is no standard method or coefficients for modeling losses through junctions for either pressure flow or open channel flow.

The attempt to identify commonly used loss coefficients led in turn to the second goal of the literature review: further examination of the references in these works, sometimes going back to the original experimental research underlying many of the methods.

As recently as 1994, one researcher commented, "A literature search showed that junction loss data for the range of flow conditions required in the design were either non-existing or so uncertain that they were useless." (Serre et al., 1994)

Lack of good junction data is a significant issue, although there were a number of studies found. In the last 50 years, many researchers have investigated different aspects of the problem. The most thorough experimentation remains that of Sangster et al (1958). Their results, however, were expressed as design charts for determining pressure losses, which are not directly useful for SWMM modeling requirements.

23.3 Loss Coefficients and Calculation Methods

One finding of this research was that there are a number of ways to calculate minor losses. Most are based on a factor applied to some variation of the velocity head at the junction of two pipes. These variations include the difference between upstream and downstream velocity heads:

$$\frac{V_u^{\,2} - \; V_d^{\,2}}{2g} \tag{23.2}$$

the upstream head alone

$$\frac{V_u^{\,2}}{2g} \tag{23.3}$$

or the downstream head alone

$$\frac{V_d^{\,2}}{2g} \quad 23.4)$$

In at least one case (King and Brater, 1963), tables of loss coefficients are keyed to the velocity head in the smaller pipe: upstream for expansions and downstream for contractions.

Several researchers (such as Sangster, et al, 1958) have made a distinction between calculation of head (energy) losses and calculation of pressure (HGL) losses. A pressure coefficient applied to the velocity through a junction outlet will provide direct calculations of the hydraulic grade line (HGL) in the upstream pipes, taking into account differences in velocity and water surface elevation changes caused by different pipe diameters. These coefficients are not suitable for EXTRAN, which calculates the HGL internally. What is desired is an energy loss coefficient, which describes only the head loss through the junction, without reference to different pipe sizes.

It is relatively straightforward to convert pressure change coefficients to energy change coefficients. Marsalek (1985) derived an equation for circular pipes from definitions of coefficients and junction geometry to make this conversion:

$$K_p = K_e + 1 - \left(\frac{D_u}{D_d}\right)^4 \tag{23.5}$$

Conversion of the pressure change coefficient to an energy change coefficient is done with a minor manipulation of this equation:

$$K_e = K_p + \left(\frac{D_u}{D_d}\right)^4 - 1 \tag{23.6}$$

For pressure flow, it is also relatively straightforward to convert coefficients applied to upstream velocity head to downstream loss coefficients. In this case, the relative size of the pipes is used, along with continuity, to show

$$\frac{K_u}{K_d} \approx \left(\frac{D_u}{D_d}\right)^4 \tag{23.7}$$

These conversions will be used against published loss coefficients and research findings throughout this chapter to normalize all the data to a single format. The most convenient to use will be a single coefficient applied to the velocity head exiting the junction. Since EXTRAN applies losses to conduits, not junctions this approach could be used by applying all the junction loss coefficients as a single factor on the downstream conduit.

23.4 Coefficients for Pressure Flow in Pipes

Minor loss calculations and coefficients for pressure flow are reported in many sources – several can be traced back to the same original research. Much of this research dates back to the early 1900s, when estimates of losses in water distribution systems motivated the studies. These are the coefficients most often shown in the design criteria and handbooks.

23.4.1 Entrance Losses

Entrance losses deal with the situation where water enters a submerged pipe from a reservoir. For this situation, the equation relating losses to downstream velocity is used:

$$h_L = K\frac{V_d^{\,2}}{2g} \tag{23.8}$$

Entrance loss coefficients are based on the shape of the entrance pipe, as shown in Table 23.2 (Task Force, 1965). These, or similar coefficients, have been widely reproduced (Merritt, 1983, Daugherty et al 1985, King and Brater, 1963). The equation where a free flowing stream enters a pipe is similar to that of a culvert entrance. The coefficients for this situation are given in a later section.

Table 23.3 Entrance loss coefficients for pressure flow .

Entrance Type	K
Inward Projecting	0.80
Sharp-Cornered	0.50
Slightly Rounded	0.25
Bell-Mouthed	0.05

23.4.2 Exit Losses

Exit losses occur when the pipe system discharges into a lake or other receiving water which is large enough so it is essentially still with respect to the discharge. In this case, the entire velocity head is dissipated, and the loss coefficient is 1.0, applied to the velocity of the upstream pipe. The situation where a pipe discharges to a free-flowing stream will be considered in the section on free-surface flow.

23.4.3 Transitions

Transitions consist of expansions and contractions in the size of the pipe. Estimating losses for transitions in pressure flow was one of the earliest fluid mechanics problems.

Expansion

The theoretical value for head loss through a sudden enlargement can be derived from the energy and momentum equations:

$$h_L = \frac{(V_1 - V_2)^2}{2g} \tag{23.9}$$

Expressed as a coefficient on the downstream velocity, this becomes

$$h_L = K\frac{(V_d)^2}{2g} \tag{23.10}$$

where:

$$K = \left[\left(\frac{D_d}{D_u} \right)^2 - 1 \right]^2 \tag{23.11}$$

One of the most widely used procedures for calculating losses in pressure flow sudden expansion transitions is described in King and Brater (1963), whose tables have been reproduced in several texts and design manuals including HEC-22, Linsley and Frazini (1975) and UWRRC (1992), showing the loss based on velocity in the upstream pipe.

The tables were created by Horace King (King and Brater, 1963) using an equation for these types of losses derived from experimental results by Archer (1913).

$$h_L = 1.1 \frac{(V_1 - V_2)^{1.92}}{2g} \tag{23.12}$$

The tables give head loss, h_L, and K based on the ratio of upstream and downstream pipe diameters. The data can't be applied directly in EXTRAN because the exponent of the velocity component is not 2.0.

As an alternative, the theoretical equation gives results reasonably close to the tables derived from Archer's equation. For the likely range of storm drain transitions ($D_d/D_u < 2.0$) the theoretical curve underestimates Archer's values for K by more than 10 percent at velocities under 4 fps and overestimates the values for velocities by more than 10% at velocities over 15 fps. If the modeler anticipates high velocity flows, reducing the theoretical value of K by a factor of 0.9 gives a reasonable approximation to the tabular values.

Contraction

The theoretical value for head loss in sudden contractions has also been determined in the early 1900s by analysis to be

$$h_L = \left(\frac{1}{C_c} - 1 \right)^2 \frac{V_d^{\,2}}{2g} \tag{23.13}$$

which results in a loss coefficient of

$$K = \left(\frac{1}{C_c} - 1 \right)^2 \tag{23.14}$$

The contraction coefficient C_c has been experimentally determined by several researchers beginning in the late 1800s. An equation for it was derived and given in King and Brater (1963). The results match the experimental data fairly closely for higher velocities.

$$C_c = 0.582 + \frac{0.0418}{\left(1.1 - \dfrac{D_d}{D_u} \right)} \tag{23.15}$$

King derived similar tables for head loss factors for contraction as those for expansion by smoothing the graphed results of several researchers' data, with the loss coefficient varying based on both the ratio of pipe sizes and flow velocity. These, like the tables for expansions, have been widely reproduced, including in AISI (1980) and UWRRC (1992).

Fluid mechanics texts and handbooks are another source of these factors. Daugherty (1985), Merritt (1983) and Linsley and Frazini (1955) provide information which can be used to derive head loss factors or contraction coefficients based on the ratio of upstream to downstream diameters or cross-sectional area. These match reasonably closely withKing and Brater's table for V = 20 cfs, as shown in Table 23.4. For pressure flow in storm drains, theKing and Brater values are recommended.

Table 23.4 Head loss coefficients for contraction in pressure flow.

D_u/D_d	1.1	1.2	1.25	1.4	1.6	1.8	2.0	2.2	2.5	3.0
Daugherty	.06		.15	.22	.28		.33		.36	
Linsley			.10		.30				.40	
Merritt	.06	.10		.22	.27	.32		.34		.38
King	.05	.09		.18	.25	.31	.33	.35	.37	.39

Bends

Similarly to closed conduit transitions, losses in bends have been studied extensively. Most of the researchers have focused on small pipes with bends of 90°, suitable for calculating water distribution or plumbing losses. Losses were found to vary depending on the ratio of the bend radius to the pipe diameter, R/d. Two sources of original research were identified: Crane (1965) and Anderson and Straub (1948).

A few researchers have worked with these data to apply them to large pressure conduits, among them King and Brater (1963), USBR (1977), and Creager and Justin (1950). USBR (1977) and Creager and Justin (1950) have both analyzed available research and have come up with substantially the same results. The King and USBR data area both based on the Anderson/Straub (1948) results.

Table 23.5 Loss coefficients for 90° bends.

R/d	Crane	Anderson/ Straub	King / Brater	USBR
1	.50	.23	.23	.23
2	.30	.14	.13	.13
4	.25	.09	.08	.09
6	.15	.08	.08	.07
8	.15	.08	.08	.07

Smaller bend angles have proportionately smaller loss coefficients. Again, these have been reported frequently with good agreement in a number of handbooks, including King and Brater (1963), USBR (1977), and Creager and Justin (1950). King and Brater (1963) reported two sets of these factors, one by Fuller, which was an approximation, and one by USBR. The USBR factors, derived from a study of large conduits, are probably the most suitable for use in calculating storm drain losses.

Hinds (1928) cited in Merritt (1983), recommended adjusting the losses for other bend angles with the following equation. Results are shown in the last column of Table 23.6.

$$K = \left(\frac{\alpha}{90}\right)^{0.5} \tag{23.16}$$

This equation matches the USBR curve closely for angles between 45^o and 90^o but will overestimate losses for more shallow angles.

Table 23.6: Loss coefficient reduction factors for other bend angles.

Angle	USBR	Fuller	Creager and Justin	$(a/90)^{0.5}$
22.5	.42	.50	.45	.50
45	.70	.75	.70	.71
60	.83		.85	.82
90	1.00	1.00	1.00	1.00

23.5 Coefficients for Free Surface Flow in Open Channels

23.5.1 Transitions in Size or Shape

Two equations for entrance and exit losses were widely reported:

for flow contraction:

$$h_L = 0.1\frac{V_u^{\ 2} - V_d^{\ 2}}{2g} \tag{23.17}$$

for flow expansion:

$$h_L = 0.2\frac{V_u^{\ 2} - V_d^{\ 2}}{2g} \tag{23.18}$$

These were cited in ASCE (1961), UDFCD (1969), and AISI (1980), where they were attributed to Hinds (1928). They were also cited in French (1985), attributed to Chow (1959).

For rectangular channel transitions, the recommended angle of the transition section is 12.5 degrees, cited in ASCE (1961), Daugherty (1985), and French (1985).

All of these references eventually can be traced back to the work of Hinds (1928), which was a study of transitions in trapezoidal and rectangular open channels. They are based on well-designed transitions in flumes, which are smoother than typical storm drain transitions.

Loss coefficients for rectangular open channels were subsequently developed by Chow which vary depending on the shape of the transition structure as shown in Table 23.7. These appear to be the best researched coefficients for transitions in man-made channels.

All of the coefficients in this table are based on the differential in velocity head and thus can’t be applied directly to the downstream flow only. The equation can be normalized using the cross-sectional area for each section, with the continuity equation used to equate Q at both sections:

$$h_L = K\frac{V_u^{\,2} - V_d^{\,2}}{2g} \tag{23.19}$$

$$h_L = K\left(\frac{A_d^{\,2}}{A_u^{\,2}} - 1\right)\frac{V_d^{\,2}}{2g} \tag{23.20}$$

Table 23.7 Transition loss coefficients for rectangular open channels.

Type of transition	Inlet	Outlet
Warped	0.10	0.20
Cylinder-quadrant	0.15	0.25
Simplified straight line	0.20	0.30
Straight line	0.30	0.50
Square-ended	0.30+	0.75

23.5.2 Curves or Bends

Unlike the situation for calculating transition losses, there do not appear to be any agreed upon equations and coefficients for bend losses in open channel flow. Handbooks and design criteria use different methods, usually without attributing any research or studies to back them up. Where there is concurrence, it us usually because the coefficients for pressure flow have been used.

There have been several studies of bend losses in open channels, however. The most thorough analysis, and the best source of energy loss coefficients is a study by Shukry (1950) whose paper investigated the characteristics of spiral flow in bends in rectangular channels. Shukry found that bend losses were a function of the following variables:

Re	Reynold's Number
θ	Angle of curvature
R/b	Ratio between curve radius, R, and channel width, b
Y/b	Ratio between flow depth, y, and channel width, b

The routines in EXTRAN do not currently allow for a loss coefficient that changes dynamically with velocity (represented by Re) or depth, so a simplified method of estimating a loss coefficient from Shukry's work should begin with θ and R/b, both of which are functions of channel geometry, then check to see if differences from velocity and depth are significant.

Shukry's recommendation was to fix Re, find R/b for the section, find K using the curves he had developed, then make corrections for the curve angle and depth. Since his findings showed that losses for high bend ratios are negligible at any angle, this approach appears to be sound. Shukry presented his results in a series of charts comparing two of the variables against the loss coefficient in a family of curves, with measured data points shown.

Derivation of Bend Loss Coefficients from Shukry's Study

The analysis which follows is based on the data points alone, read from the charts. Points in bold shown in the tables that follow were measured by Shukry, the others have been interpolated. Using data from the chart with Re fixed at 31,500, the following measurements and interpolations were made, shown in Tables 23.8 – 23.11. Review of Shukry's results shows that in typical open channel systems, bend losses are negligible when the curve angle is less than 45 ^{o} and the R/b ratio is greater than 3.0.

Table 23.8: Shukry Data for y/b = 1.20

	R/b			
θ/180	0.50	1.00	2.00	3.00
1.00	.82	**.25**	.07	.01
.75	.75	**.23**	.06	.01
.50	**.72**	**.20**	**.08**	**.01**
.25	.07	**.02**	.01	.00
.125	.00	**.00**	.00	.00

Table 23.9 Shukry Data for y/b = 1.00.

	R/b			
θ/180	0.50	1.00	2.00	3.00
1.00	.95	**.29**	.08	.01
.75	.88	**.27**	.07	.01
.50	**.90**	**.22**	**.04**	**.01**
.25	.10	**.03**	.01	.00
.125	.00	**.00**	.00	.00

Table 23.10 Shukry Data for y/b = 0.80.

	R/b			
θ/180	0.50	1.00	2.00	3.00
1.00	1.11	**.34**	.09	.01
.75	1.05	**.32**	.09	.01
.50	**.95**	**.29**	**.08**	**.01**
.25	.13	**.04**	.01	.00
.125	.00	**.00**	.00	.00

Table 23.11: Shukry Data for y/b = 0.60

	R/b			
θ/180	0.50	1.00	2.00	3.00
1.00	1.28	**.39**	.11	.01
.75	1.21	**.37**	.10	.01
.50	**1.10**	**.32**	**.09**	**.01**
.25	.16	**.05**	.01	.00
.125	.00	**.00**	.00	.00

To use the experimental results most effectively in EXTRAN, it would be ideal to develop an equation relating the loss coefficient to channel geometry. There are no simple equations that reproduce all the values in the table. Boundaries were placed on the applicability of the equations based on typical situations for storm drains. Simplifying assumptions were also made. First, because EXTRAN does not vary the head loss with depth, an assumption was made that no change would be made based on the variable y/b. For the equation to be developed, y/b was assumed to be 0.60, which gives the highest losses. The assumptions include:

r/B > 1.0
Angle between 45 and 90 degrees
y/b = 0.60

Following the procedure used by USBR (1977), we first derive an equation relating the loss coefficient to the r/B ratio for a bend angle of 90 degrees. There is no simple form which reproduces the values exactly. A reasonable approximation can be made with the following equation, for values of r/b > 1.0:

$$K_1 = e^{-1.2\frac{r}{B}} \tag{23.21}$$

The result of this equation compared to Shukry's data is shown in the following table:

Table 23.12: Bend loss coefficients derived from Shukry's data.

	R/b		
y/b	1.00	2.00	3.00
1.2	.20	.08	.01
1.0	.22	.04	.01
0.8	.29	.08	.01
0.6	.32	.09	.01
Avg	.26	.07	.01
Avg 0.6 and 0.8	.31	.09	.01
Eqn	.30	.09	.03

The results of the equation are reasonably close to the average of the losses at all depths, and quite close to the losses for the lower depth ratios most likely to be found in drainage systems. They are too high for the r/b ratio of 3.0, but drop off at higher ratios, just as Shukry's data did.

The next step with Shukry's data is to make an adjustment for the bend angle. Again, this is done by making an approximation based on the measured data. There are three data points in the range commonly found in storm drains: 22.5, 45 and 90 degrees. There are three data points for extreme bends where the flow doubles back: 90, 135, and 180 degrees. When graphed, there is a distinct break point at 90 degrees, with a linear relationship on either side. There is also a distinct breakpoint for angles less than 45 degrees.

For angles between 22.5 and 45 degrees, the following equation is a linear approximation:

$$K_2 = 0.005\theta - 0.11 \tag{23.22}$$

For angles between 45 and 90 degrees, the following equation is a linear approximation:

$$K_2 = 0.019\theta - 0.71 \tag{23.23}$$

For angles larger than 90 degrees, this equation is a linear approximation:

$$K_2 = 0.003\theta + 0.77 \tag{23.24}$$

Figure 23.1 shows a comparison between Shukry's data and these equations for y/b = 0.6.

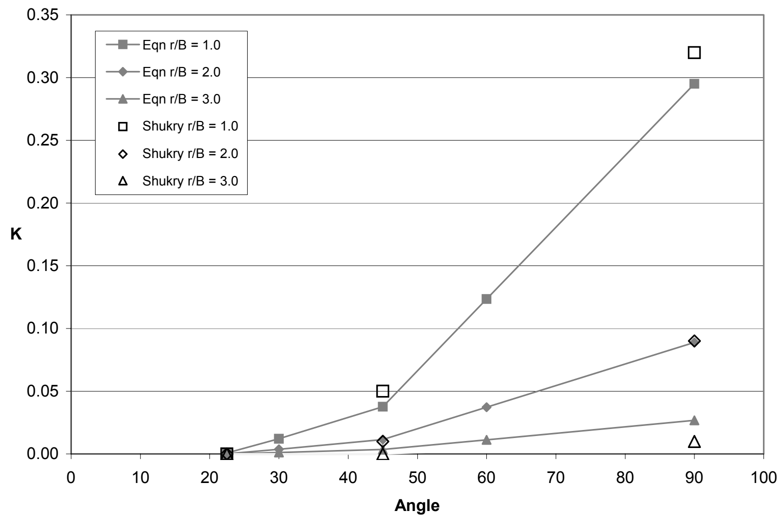

Figure 23.1 Comparison of derived equation with Shukry's data.

23.5.3 Culvert Entrance and Exit

The accepted standard for losses at culverts is FHWA (1985): *Hydraulic design of highway culverts* which is an update of US Bureau of Public Roads or Federal Highway Administration documents for culvert design and calculation of losses.

Table 23.13 Culvert loss coefficients (USBPR, 1961).

Type of Structure and Design of Entrance	k_e
Pipe, Concrete	
Mitered to conform to fill slope	0.7
* End-Section conforming to fill slope	0.5
Projecting from fill, sq cut end	0.5
Headwall or headwall and wingwalls	
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Socket end of pipe (groove-end)	0.2
Projecting from fill, socket end (groove-end)	0.2
Beveled edges, 33.7E or 45E bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Mitered to conform to fill slope, paved or unpaved slope	0.7
Headwall or headwall and wingwalls square-edge	0.5
* End-Section conforming to fill slope	0.5
Beveled edges, 33.7E or 45E bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Wingwalls at 10E to 25E or 30E to 75E to barrel	
Square-edged at crown	0.5
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30E to 75E to barrel	
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Side- or slope-tapered inlet	0.2

Virtually all of the handbooks and design criteria reviewed use these coefficients for culvert losses. Table 23.13 shows values for various end sections and pipe materials.

This publication makes the assumption that the exit loss coefficient is 1.0, meaning that all the velocity head is lost. This is appropriate for discharges to ponds or lakes, but if the culvert discharges to a free flowing stream or channel, the modeler should consider using the coefficients for transition losses instead.

The minimum loss coefficient that should be used is 0.30, which is Chow's value for simplified straight-line transitions. It should be adjusted for change in cross-sectional area as discussed earlier.

23.6 Coefficients for Pressure Flow in Junctions

Though there are many types of junctions with many combinations of pipes entering and leaving, researchers have concentrated on a subset of those that meet in a T, listed in order of complexity. Research which has addressed these aspects of junction flow is summarized in Table 23.14.

- straight flow through a junction,
- flow turning 90° in a junction,
- lateral on a through main with different proportions of flow in the lateral, and
- opposed laterals.
-

Table 23.14 Research results reported for junction losses (pressure flow)

	Sangs ter	Marsalek	Hare	Johns ton	Pedersen	Blaisdell	Townsend	Serre
Straight through	x		x	x	x			
90^o turn	x	x	x					
Lateral on through main	x	x		x		x	x	x
Opposed laterals	x	x						
Manhole benching	x	x		x	x			
Inlet	x							

Most researchers have concentrated on methods of reducing losses through benching or shaping the invert to guide the flow and developed calculation procedures as a secondary goal to their studies. Several have conducted a dimensional analysis to identify the independent variables governing junction losses.

Between this analysis and the experimental results, a consensus can be seen on which independent variables have the most effect on junction losses. During experimental runs, many flow and geometric parameters were varied; however, not all researchers varied all the parameters or the same ones.

All found that there is no single loss coefficient that applies to every flow entering and exiting a junction. Instead, junction geometry plays an important role and K will vary depending on the size and configuration of the pipes and junction. The most important variables are:

- ratio of junction width to the downstream pipe diameter (b/D_d),
- ratio of upstream and downstream pipe diameters (D_u/D_d), and
- benching or invert shaping.

23.6.1 Straight Flow through a Junction

Four studies were found which developed experimental measurements of head loss in straight-through manholes (two-pipe junctions with neither lateral inflows nor bends). All three independent variables were studied; however, none of the studies varied all three. Variations of D_u/D_d and b/D_d were studied by Sangster et al. (1958), while b/D_d and invert benching were varied in research by Marsalek (1984), Johnston and Volker (1990) and Pedersen and Mark (1990).

Sangster et al. presented results in two forms. The first was a set of experimental results from 15 tests varying D_u/D_d and b/D_d with one curve drawn to fit the data, and second was as a set of curves to be used for design. Table 23.15 shows these values. The design column (1) uses data for b/D_d = 1.0. For comparison, values for sudden expansion and contraction are shown in column (3), taken fromKing and Brater's tables for V = 15.0 fps. This velocity was chosen to match the Froude numbers of Sangster's study at typical diameters for storm drain systems; 24" to 108".

These results are shown two ways in the table. Columns (1) (2) and (3) show the raw data. Columns (4) (5) and (6) show the coefficients normalized as energy loss coefficients applied to the downstream velocity head. The values become more similar as the difference between pipe sizes is greater, indicating that manholes with a large disparity in pipe sizes act somewhat like sudden expansions and contractions. One can also note that Sangster's design data are very close to the fitted experimental curve.

Table 23.15 Variation of upstream and downstream pipe diameter.

D_u/D_d	D_d/D_u	(1)	(2)	(3)	(4)	(5)	(6)	(7)
2.50	0.4	1.4		0.38	0.43		0.43	
2.00	0.5	1.3		0.34	0.36		0.34	
1.67	0.6	1.2		0.27	0.33		0.27	
1.43	0.7	1.00	1.05	0.19	0.24	0.29	0.19	
1.25	0.8	0.80	0.83	0.11	0.21	0.24	0.11	
1.11	0.9	0.45	0.44	0.04	0.16	0.15	0.04	
1.00	1.0	0.00	0.07	0.00	0.00	0.07	0.00	0.20
0.9	1.11	-0.45	-0.44	0.05	0.07	0.08	0.08	0.32
0.8	1.25	-1.10	-1.10	0.12	0.34	0.34	0.19	0.49
0.7	1.43	-2.10	-2.10	0.25	1.08	1.08	1.04	1.23
0.6	1.67	-3.60	-3.55	0.37	3.12	3.07	2.87	
0.5	2.00	-6.00		0.51	9.00		8.16	

(1) K_p, Sangster, et al., fitted curve to experimental data
(2) K_p, Sangster, et al., design charts
(3) K_e, King and Brater Tables 6-7 and 6-10 for v=15.0 fps'
(4) K_e for velocity head, Sangster, et al., fitted curve
(5) K_e for velocity head, Sangster, et al., design charts
(6) K_e, King and Brater, for velocity head in the downstream pipe
(7) K_e, for velocity head, Hare,

Ratio of upstream and downstream pipe diameters

For a single b/D_d, where $b/D_d = 1$, there are two sources of data for determining the effect of D_u/D_d on the loss coefficient: Sangster et al (1958) and King and Brater (1963). The latter data are for sudden expansions and contractions not associated with junctions, however. Hare (1983) reported similar results for $b/D_d = 2$. Table 23.15 shows these results. The first columns show the data reported in the research, and the last columns show the data normalized as energy loss coefficients applied to the velocity head in the downstream pipe. The data from Sangster et al (1958) track closely with the equation developed by King. The Hare data are somewhat higher, but include potential losses from the differing manhole geometry.

The similarity between the equation developed by King and Sangster's experimental results is relatively close for contractions of less than 0.7 and expansions of greater than 1.25. The use of Archer's equation in this range seems appropriate for transitions in junctions.

In the narrower range typical of most storm drain junctions, the Archer equation gives loss coefficient values lower than the experimental results.

The results of the comparison show that the values developed by King for expansion and contraction in closed pipes (column 3) is a good approximation for the losses attributable only to the change in pipe diameters in flow through a junction.

Ratio of junction width to the downstream pipe diameter

Effects of junction width (b/D_d) for D_u/D_d = 1.0 were studied by all the researchers cited and all found the same basic linear relationship between b/D_d and K with a shape factor, α, which is an empirical factor based on experimental measurements.

Review of Sangster's design curves show that junction width effects can be expressed as additions to the effects of variations in pipe size (Table 23.16), with α varying somewhat with the ratio of pipe sizes, again trending lower as pipe size disparity grows. The other researchers found α to be about 0.10 or 0.12 (See Table 23.17).

Table 23.16 Effects of junction width and pipe size on K_e (Sangster et al 1958).

	b/Dd			
Dd/Du	1.0	2.0	3.0	α
0.8	0.24	0.29	0.34	0.05
0.9	0.15	0.22	0.29	0.07
1.0	0.07	0.14	0.21	0.07
1.11	0.08	0.15	0.21	0.07
1.25	0.34	0.39	0.44	0.05
1.43	1.08	1.10	1.12	0.02

Table 23.17 Shape factor relating b/D_d to K.

Source	α
Marsalek (1984)	0.12
Hare reported by Johnston and Volker (1990)	0.10
Johnston and Volker (1990)	0.10
Pedersen and Mark (1990)	0.12

The experimental results show that this factor should be added to the loss coefficient for pipe size transitions as an additional parameter for the overall junction loss coefficient.

$$K_2 = 0.10\frac{b}{D_d} \tag{23.25}$$

Benching or invert shaping

Four of the researchers varied types of invert shaping to estimate the effects. All agreed that benching/shaping the manhole invert was the most significant factor affecting losses in any type of junction. The most commonly studied methods of invert shaping were to add a bench to guide the flow at half the depth and at the full depth of the entering/exiting pipes. For two-pipe systems (straight-through flow and 90° turns within a manhole) the ratio of junction width to downstream pipe diameter (b/D_d) was important, as was the ratio of upstream and downstream pipe diameters (D_u/D_d). Insignificant or minor factors included manhole shape (square, round, or rectangular), depth of submergence, Froude number, and Reynolds number.

For three-pipe systems, the ratio of flows between lateral and upstream mains (Q_u/Q_b) and the ratio of the lateral and downstream pipe diameters (D_b/D_d) were also significant. Total flow was varied by Sangster (1958) and found to have no effect on losses.

The experimental results of benching clearly show a reduction in head losses. Unfortunately, none of the studies varied all three of the most important variables in a single experiment. The most comprehensive results of benching were measured when upstream and downstream pipe sizes were the same.

Table 23.18 Reduction factors for invert benching.

Source	None	Half	Full
Pedersen and Mark (1990)	1.00	0.58	0.21
Johnston and Volker (1990)	1.00	0.60	0.56
Marsalek (1984)	1.00	0.70	0.52

The effects of benching were studied for $D_d/D_u = 1.0$, where they showed a clear reduction in the amount of head loss and the loss coefficient. Table 23.18 presents the results as a factor that can be applied to the loss coefficient where $D_d/D_u = 1.0$. Extrapolations to other pipe diameter ratios may not be valid. It should be noted that these values have been developed by the author, averaging results for different velocities or manhole shapes.

The resulting contribution to determining K for a junction is to multiply the earlier value by a reduction factor, K_3. A value of 0.60 is representative of the findings of all three researchers for either the full or half-benched condition, with the exception of Pedersen and Mark (1990) for full benching.

Computation of Junction Loss Coefficient

Summarizing the research and discussion, there are three factors that enter into calculating a loss coefficient for a junction in a storm drain network where the flow goes straight through, with no laterals or change in direction. The calculation takes the form of

$$K_j = (K_1 + K_2)K_3 \tag{23.26}$$

where K_3, the loss due to expansion / contraction, is taken from King and Brater's values for V = 15 fps (Table 23.15, column 6) and

$$K_2 = 0.10\frac{b}{D_d} \tag{23.27}$$

represents the loss due to difference in pipe and manhole size, and

$$K_3 = 0.60 \tag{23.28}$$

represents the reduction in loss due to benching the invert.

23.7 Coefficients for Free Surface Flow in Junctions

Experimental results for free surface flow in junctions were difficult to find. Most researchers have concentrated on estimating losses in sanitary sewer systems where pressure flow is a more frequent occurrence and where failures can have more serious health consequences.

23.7.1 Straight Flow through a Junction

Marsalek (1985) conducted experiments for both pressure flow and free surface flow and reported differing values depending on manhole construction and pipe diameter. Losses were calculated using the equation based on the downstream velocity head. Loss coefficients were given for circular manholes in a graph, which showed W/d vs. K related as follows.

Table 23.19 Coefficients for straight flow through a junction (free flow).

W/d	No shaping	Half benching	Full benching
2.3	0.29		0.12
2.0	0.22	0.16	
1.6	0.16		
1.3	0.13		
1.0	0.12		

23.7.2 Flow Turning in a Junction

Marsalek (1985) gave values of loss coefficients for 90 degree bends in manholes. Again, these are based on size of the manhole, diameter of pipe, and amount of invert shaping or benching. The values for free-surface flow are approximately 2/3 of the values he found for submerged flow:

Table 23.20 Coefficients for 90 degree bend through a junction (free flow).

W/d	No shaping	Half benching	Full benching
2.3	1.1	1.1	0.7

It is assumed that these coefficients combine all entrance, bend, and exit losses into one number.

23.8 Conclusion and Recommendations

The purpose of this chapter was to review commonly used coefficients for modeling minor losses in storm drains using EXTRAN, to find experimental verification to determine if they are being used in the appropriate situations, and if not, to develop better coefficients based on experimental data. The literature review found much more information for pressure flow scenarios than for free surface flow. There was also more information for transitions in pipes than for transitions through junctions such as manholes.

The first recommendation from the chapter is to urge researchers to conduct new experimental studies with the goal of developing loss coefficients that can be used for better estimates of junction losses in pipes with free surface flow.

The review also recommends that modelers should use the coefficients and methods of estimating them shown in Table 23.21, which have been developed in this review from the best available experimental results available.

Table 23.21 Recommended coefficients.

Type of Loss	Source	Table or Eqn
Pressure Flow		
Entrance	Merritt	Table 23.3
Exit		Assumed = 1.0
Expansion and Contraction	King	Table 23.4
Bends	USBR	Table 23.5
Free Surface Flow		
Expansion and Contractions	Chow	Table 23.7
Bends	Derived from Shukry	Eqns 23.22-23.24
Culvert entrance	FHWA	Table 23.13
Pressure Flow, Junctions		
Straight through	King	Table 23.15 Col 6
Expansion and Contraction	King	Table 23.15 Col 6
Bends		
Free Surface Flow, Junctions		
Straight through	Marsalek	Table 23.19
Bends	Marsalek	Table 23.20

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