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MANHOLES HEAD LOSSES IN DRAINAGE HYDRAULICS

by

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MANAGEMENT PERSPECTIVE

Organizers of the 21st Congress of the International Association for Hydraulic Research, requested specific information on hydraulic losses at junctions in storm sewers when operating in a surcharged condition. This paper, using data developed for a client, provides specific information on these losses and provides methods to reduce such losses. Head losses, which are unnecessarily large owing to poor design, increase the costs of excavation or increase flooding frequency. A typical practical example is given. All sewer system designers should adopt the benching designs indicated in the report.

T. Milne Dick
Chief
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PERSPECTIVE-GESTION

Les organisateurs du 21e congrès de l'Association internationale de recherches hydrauliques ont exprimé le désir d'obtenir des renseignements précis sur les pertes d'énergie dans les raccordements d'égouts pluviaux en régime d'écoulement forcé. Le présent rapport se fonde sur des données relevées pour un client. Il décrit de façon détaillée les pertes d'énergie précitées et propose des moyens pour les réduire. Les pertes d'énergie, qui peuvent être considérables, sont liées à une construction mal pensée. Elles entraînent une hausse des coûts de terrassement ou une augmentation de la fréquence des inondations. Nous en donnons un exemple pratique typique. Les constructeurs de banquettes d'égout auraient intérêt à adopter les normes que nous proposons dans le présent rapport.

Le chef,
T. Milne Dick
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RÉSUMÉ Il ressort de l'examen des données récentes sur les pertes d'énergie aux raccordements d'égout que, pour les raccordements dont les conduites sont relativement bien alignées, la configuration des banquettes est le facteur géométrique le plus important. Par conséquent, on a classé les données obtenues de sources diverses en quatre groupes correspondant chacun à un type de banquette particulier. Ces données peuvent être utilisées pour diriger l'écoulement à l'intérieur d'un système d'égout en régime forcé. Nous avons utilisé le modèle EXTRAN à cette fin. Bien que le modèle ne tienne pas compte explicitement des pertes d'énergie dues à la configuration des raccordements, il est possible de simuler ces pertes en leur substituant la rugosité des parois de conduite équivalente. Si l'on intègre les pertes d'énergie aux raccordements aux variables d'analyse du modèle, les résultats que l'on obtient sont plus près de la réalité et révèlent que la capacité du système est surestimée, que le profil hydraulique est plus élevé dans certaines parties du système et que le débit des eaux pluviales sortant des regards est plus important lorsque les raccordements sont complètement noyés.

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Manhole head losses in drainage hydraulics

SUMMARY A review of recent data on head losses at sewer manholes indicates that for individual junction types with favourable pipe alignments, the most important geometric factor affecting the head loss magnitude is the benching inside the manhole. Consequently, the head loss data from various sources were classified according to the benchings into four basic groups. These data can then be readily used in flow routing in surcharged sewer systems. For this purpose, the EXTRAN routing model was used. Although the model does not allow an explicit consideration of junction head losses, such considerations can be accomplished indirectly by using the equivalent pipe roughness. The inclusion of junction head losses in the analysis leads to more realistic computational results which indicate a lower system capacity, higher elevations of the hydraulic grade lines in some parts of the system, and greater outflows of stormwater from the junctions which are fully surcharged to the ground surface.

1 INTRODUCTION

The hydraulic design of sewer networks is based on equations of mass continuity and energy conservation. The latter equation requires considerations of two types of head losses - friction losses in sewer pipes and minor losses at various appurtenances, among which the most common are sewer junction manholes. Such manholes are used where two or more pipes join, or the pipe diameter, grade and alignment change.

While the friction losses in sewer pipes have been extensively studied in the past and can be adequately characterized for practical purposes, only limited information is available on head losses at junction manholes. When the system is surcharged, the minor head losses caused by junctions, sewer inlets, house sewer connections and other appurtenances may become rather significant and should not be neglected.

Although the junction head losses (as well as other minor losses) should be considered in the sewer design regardless of the design method used, the importance of such considerations has increased in recent years with the introduction of computerized design methods which include the pressurized flow routing through sewer networks. It should be recognized that practically every drainage system will surcharge under some circumstances. Such surcharging may either represent the special design conditions which allow surcharging to obtain higher pipe capacities and to lower the construction costs, or it may be caused by the occurrence of a storm of a lower-than-design frequency. The surcharging of the sewer system is not necessarily harmful, as long as the hydraulic grade line does not exceed the elevation above which damages occur. Such damages, or just inconveniences, result from basement flooding, surface flooding, and combined sewer overflows. Thus, to evaluate the risk of such damages in surcharged sewers, it is necessary to calculate the hydraulic grade line elevations in individual system elements. The purpose and accuracy of such calculations is defeated by the neglect or improper consideration of junction manhole head losses.

The main objectives of the paper that follows are to present, review and classify recent data on manhole head losses and to demonstrate the use of such data in pressurized flow routing in sewer networks.

2 HEAD LOSSES AT SEWER JUNCTION MANHOLES

General problems of flows through channel junctions have been discussed by Chow (1964) who concluded that the flow through junctions was a complicated problem, which was not resolvable analytically and that the best solutions would be found through model studies. Indeed, the literature survey on this subject indicated that the studies of junction head losses by means of physical models were by far the most common. Other approaches, such as field observations (Ackers, 1959) and the applications of the momentum equation (Hare, 1983; Jensen, 1981) were not productive.

2.1 General Considerations

In experimental studies of junction manholes, it is useful to begin with a dimensional analysis of the problem. The results of such an analysis are presented below for three basic types of junction manholes - straight-flow-through manholes, manholes with bends, and T-junctions of a main with a lateral. The basic notation used in the analysis is shown in Fig. 1 for the simplest junction type studied and, furthermore, the following customary definitions are adopted:

$$\Delta E = K \frac{v^2}{2g} \qquad (1)$$

$$\Delta P = K_p \frac{v^2}{2g} \qquad (2)$$

$$\Delta H_w = K_w \frac{v^2}{2g} \qquad (3)$$

where ΔE is the energy head loss due to the junction, ΔP is the pressure change, ΔH_w is the water level change at the junction, K is the energy head loss coefficient, K_p and K_w are the

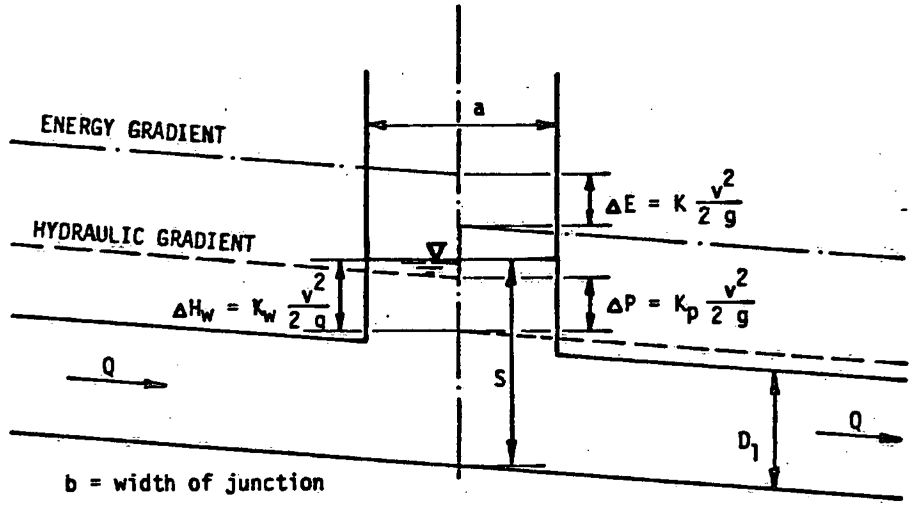

Figure 1 Notation sketch for junction manhole

coefficients of pressure and water level change, respectively, v is the velocity in the outlet pipe, and g is the acceleration due to gravity.

For a steady pressurized flow through a particular junction design, which is further characterized by the manhole base shape and the benching, the head loss coefficient K is a function of eight independent variables:

$$K = f(Q, g, \rho, \mu, a, b, S, D_1) \qquad (4)$$

where Q is the discharge, ρ is the fluid density, μ is the fluid viscosity, a is the junction length, b is the junction width, S is the junction water depth, and D_1 is the outlet pipe diameter. Dimensional analysis then yields the following expression:

$$K = f\left(\frac{v_1}{\sqrt{g\,D_1}}, \rho\frac{v_1 D_1}{\mu}, \frac{S}{D_1}, \frac{a}{D_1}, \frac{b}{D_1}\right) \qquad (5)$$

Analogous expressions, with different functional relationships, would be derived for the other two coefficients K_p and K_w.

In Equation (5), the first three independent terms are hydraulic variables and the remaining two terms describe the junction geometry.

2.1.1 Straight-flow-through junctions

The case of the straight-flow-through junctions is described by Equation (5).

The first independent term is the Froude number. As shown previously by Sangster et al. (1958), head loss coefficients are not a function of the Froude number in pressurized flow and this term may be omitted.

The second term is the Reynolds number. In practical design with specifications of the minimum pipe diameter and flow velocity, the Reynolds number will be above a certain limiting value. For example, for the minimum pipe diameter of 0.305 m and the minimum flow velocity of 0.61 m/s, the minimum Reynolds number for the full-pipe flow will be about 1.4 x 10^5 (depending on fluid viscosity). Such flow conditions may be more difficult to achieve in the laboratory tests, particularly for small pipes and flows. Even under such circumstances, it is necessary to keep the Reynolds number above a certain limit, so that the observed head loss coefficients are not unduly affected by the viscous forces. Black and Piggott (1983) recommend to keep the Reynolds number above 10^4. In another study (Marsalek, 1985), the head loss coefficients for a manhole with a 90° bend were independent of the Reynolds number for Re > 3 x 10^4. Above this value, no effects of the Re number were detected. Thus as long as the Reynolds number is maintained above a certain minimum value (say 3 x 10^4), the Reynolds number can be eliminated from further considerations.

The next term is the submergence which is defined as S/D_1. The effect of this variable on head loss coefficients is not well understood, particularly for low values of S/D_1.

de Groot and Boyd (1983) as well as Sangster et al. (1958) concluded that the pressure change coefficients of a T-junction were not affected by the submergence for S/D_1 greater than 3.0. Consequently, most of the experimental work is done for large submergence values to avoid any submergence effects. For S/D_1 < 3.0, some effects of the submergence were reported for certain types of junctions (Lindvall, 1984). From the practical point of view, there is a great interest in losses for low submergence, because such cases are often encountered in design conditions. For example, in the practical application presented in the second part of this paper, the maximum submergence ratios varied from 1.2 to 6.3. The values of this ratio are affected by the hydraulic conditions as well as by the thickness of the pipe cover and pipe diameter.

The effects of the submergence on head loss or pressure change coefficients were studied for straight-flow-through junctions (Archer et al., 1981; Lindvall, 1984; Marsalek, 1984b), manholes with a 90° bend (Marsalek, 1985), and T-junctions (Lindvall, 1984). For the last two junctions, both Lindvall (1984) and Marsalek (1985) reported no effects of the submergence for S/D_1 as low as 1.3. The only case where the submergence effects were significant and clearly detected were the straight-flow-through junctions studied by Lindvall (1984). In this case, the head loss coefficient dramatically increased because of special flow regimes which formed inside the junction for S/D_1 smaller than 2.0. The first regime was characterized by strong lateral oscillations (sloshing) and the second one by a strong vortex. The occurrence of these regimes depended on the submergence, flow rate, and detailed junction geometry. For manholes with a high benching, the effects of submergence were greatly reduced and became of secondary importance from the practical point of view. Neither of these regimes was reported by Archer et al. (1978) or Marsalek (1984b), but the presence of the vortex was also found by Howarth and Saul (1984), in somewhat different installations. It also appears that the ratio D_m/D_1 (where D_m is the manhole diameter) may be critical for the establishment of the above flow regimes. From the theoretical point of view, further investigations of the effects of low submergence on head losses at manholes with a limited benching would be of interest.

The last two terms in Equation (5) are geometric parameters of the junction manhole, defined here as the relative junction length and width. In the current Canadian practice, the use of precast manholes is widespread and, for these manholes, the range of junction dimensions can be fairly limited in the experiments. The most common precast manholes have a round base with a diameter of 1.2 m and the maximum pipe diameter of 0.61 m and the parameters a/D_1 and b/D_1 can be replaced by a single parameter D_m/D_1 which will be generally greater than 2.0 and smaller than 5.

In the range of D_m/D_1 from 2 to 5, the head loss seems to be slightly affected by the junction dimensions, with some exceptions. It is generally agreed that the head loss slightly increases with the increasing D_m/D_1 (Lindvall, 1984; Marsalek, 1984b). Some installations, however, may be susceptible to the formation of special flow regimes accompanied by high head losses. This was reported by Lindvall (1984) for $D_m/D_1 = 2.6$ and a certain range of submergence. Again, the occurrence of such phenomena can be avoided by using full benching (i.e., reaching to the pipe crown at the junction).

In summary, it appears that the head losses at straight-flow-through junctions are not affected by the flow variables provided that these variables are maintained above some critical values. Under such circumstances, the losses are influenced only by the junction geometry which can be described by the manhole base shape, the junction manhole dimensions, and the benching inside the junction. Among these three parameters, the benching has the most pronounced influence (Marsalek, 1984). Thus within the above limitation, it can be stated that the K is a function of D_m/D_1, manhole base shape, and the benching.

2.1.2 Manholes with a bend

A dimensional analysis for a particular manhole design yields results similar to those given by Equation (5) for the preceding case. With regard to the individual independent variables, it appears that the submergence ratio S/D_1 is even less important in this case, particularly for large deflection angles. For manholes with a 90° bend, Marsalek (1985) have not found any effects of the submergence for S/D_1 values as low as 1.3. Thus, as in the previous case, the head losses are controlled only by the junction geometry. Two additional geometric parameters need to be considered - the deflection angle and the alignment of pipes in junctions without any benching. The latter parameter was studied by Hare (1983) who recommended that the axes of the inflow and outflow pipes should intersect on the downstream junction wall in order to reduce the junction head loss. Only such designs are considered here.

Thus, the head loss coefficient for junctions with a bend depends on the deflection angle, the junction dimension D_m/D_1, the base shape, and the benching.

2.1.3 T-Junctions of a main with a lateral

For T-junctions, additional independent variables need to be considered. In particular, there is a new flow variable - the flow ratio Q_2/Q_1, where Q_2 is the discharge through the main pipe ($Q_3 = Q_1 - Q_2$, where Q_3 is the lateral discharge). Among the geometric parameters, two additional terms need to be considered - D_2/D_1 and D_3/D_1, where D is the pipe diameter and subscripts 2 and 3 refer to the main and lateral, respectively. Two head loss coefficients can be discerned in this case, for the main and lateral. Both of them can be expressed as functions of the flow ratio Q_2/Q_1, the manhole dimension D_m/D_1, the base shape, pipe sizes D_2/D_1 and D_3/D_1, and the benching.

2.2 Review of Recent Head Loss Data

Recent data on junction pressure changes are briefly reviewed in this section. In particular, the data pertinent to the straight-flow-through junctions, manholes with bends, and T-junctions are reviewed.

2.2.1 Straight-flow-through junctions

These junctions are typically used for maintenance purposes to allow access to the sewer system. Experimental data on such junctions were recently published by Archer et al. (1978), Hare (1983), Lindvall (1984) and Marsalek (1984b). Practically all the published data fall within the range from 0.05 to 0.30, with minor exceptions for very low submergence. By restricting the considerations to higher submergence ratios, the published data can be classified according to the following parameters: manhole benching, manhole base shape, and the relative manhole size. Among these parameters, the benching seems to be the most important parameter. Four basic benching designs are shown in Figure 2 and these are referred to as moulds M1, M2, M3, and M4.

The first mould represents the case with no benching which should produce the highest head losses. This type is rarely used in the Canadian practice because of its unfavourable operation. The second mould is obtained by extending the lower pipe half through the junction and adding horizontal benches extending from the semicircular channel to the juncion side walls. This design provides some flow guidance to the flow passing through the junction. The third mould, M3, consists of the lower pipe half, extended by vertical walls to the pipe crown elevation, where it connects to the horizontal benches extending to the junction walls. The last design M4, is similar to M3 except for expansion of the pipe diameter through the junction. This design would be used in manholes with bends.

For the common straight-flow-through junctions, Table I lists the recommended head loss coefficients.

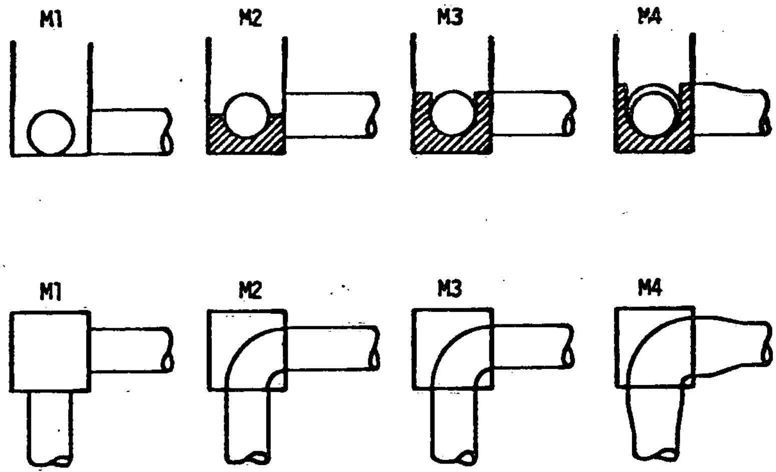

Figure 2 Junction benchings (moulds) tested

TABLE I

COEFFICIENTS FOR STRAIGHT-FLOW-THROUGH JUNCTIONS

	Mould		
	M1	M2	M3
$K(=K_p)$	0.15 - 0.30	0.10 - 0.12	0.05 - 0.15

The experimental data available further suggest that the lower values should be used for low D_m/D_1 ratios (≈2) and the upper values should be used for higher D_m/D_1 values. Where the downstream pipe diameter changes, further adjustments of the above coefficients are recommended by Hare (1983).

2.2.2 Manholes with bends

Recent studies of junctions with bends were reported by Archer et al. (1978), Hare (1983), and Marsalek (1985). Hare's data refer to the deflection angles of 30° and 60° and manhole benchings similar to mould M1. The flow deflection is achieved gradually within a relatively wide manhole. Marsalek's data refer to the deflection of 90° and all four moulds shown earlier in Figure 2. Table II lists the recommended values, for various moulds.

The data listed in Table II indicate strong influences of the angle of deflection and the benching. For the maximum angle of 90°, substantial reductions of head losses can be achieved by full benching M3 and, in special cases where even further loss reductions are desirable, this can be achieved by using mould M4. In this mould, the pipe diameter is increased at the junction by about 33%. This is achieved by two special fittings - an expander upstream and a reducer downstream of the junction. Such special fitting would increase the cost of the installations, but this may be acceptable in critical cases. Further interpretations of various data on junctions with bends are given by Black and Piggott (1983).

2.2.3 Junctions of a main with a perpendicular lateral

Recent data on these junctions have been published by deGroot and Boyd (1983) and Lindvall (1984). Such data are shown here in accord with the original references as pressure change coefficients K_p.

TABLE II

HEAD LOSS COEFFICIENTS FOR JUNCTIONS WITH BENDS ($D_1 = D_2$)

	Mould							
	M1			M2			M3	M4
Deflection angle	22.5°	45°	90°	30°	60°	90°	90°	90°
$K = K_p$	0.30	0.60	1.85	0.45	0.90	1.60	1.10	0.55

Both studies confirmed that K_p was not affected by the submergence, but only by the flow ratio Q_2/Q_1 and the benching. deGroot and Boyd's data refer to mould M1, Lindvall's data refer to moulds M2 and M3. Both sets of data are listed, with minor changes, below.

TABLE III

PRESSURE CHANGE COEFFICIENTS FOR MAIN WITH PERPENDICULAR LATERAL ($D_1 = D_2$)

Flow Ratio	Pressure Change Coefficient K_p					
	M1		M2		M3	
Q_2/Q_1	K_2	K_3	K_2	K_3	K_2	K_3
0.0	2.1	2.1	1.6*	1.6*	1.1*	1.1*
0.2	1.8	1.9	1.6	1.5	1.3	1.2
0.4	1.5	1.5	1.5	1.5	1.3	1.2
0.6	1.0	1.1	1.2	1.2	1.0	1.0
0.8	0.5	0.6	0.7	0.7	0.6	0.6
1.0	0.1	0.2	0.1*	0.1*	0.05*	0.05*

* Extrapolated data

Again, the effect of the mould is quite obvious. By providing good flow guidance through the junction, the losses and pressure changes can be significantly reduced. The coefficients vary extensively with the flow ratio Q_2/Q_1. The largest values of K_p are found for the case where all the flow is conveyed by the lateral and deflected at the junction ($Q_3 = Q_1$). This case is similar to the manholes with deflections discussed in the preceding section. The data for $Q_2/Q_1 = 0$ and moulds M2 and M3 were extrapolated from section 2.2.2. The smallest K_p's are found for $Q_2 = Q_1$ when all the flow is conveyed by the main pipe and there is no lateral inflow. This case is similar to that discussed in section 2.2.1 and the values for $Q_2/Q_1 = 1.0$ and moulds M2 and M3 were adopted from that section.

3 Applications of Junction Head Losses in Drainage Design and Analysis

Head losses at sewer junctions were considered in a study of runoff simulations in an urban test catchment (Marsalek, 1984a). The study dealt with the derivation of a urban runoff frequency curve from runoff simulations for actual storms. It was noted that for some of these storms, the sewer system surcharged and, therefore, it was desirable to simulate the pressurized flow in the catchment sewer network. Such simulations should include minor head losses at sewer junctions, which may become appreciable in a surcharged system. For runoff simulations, the Storm Water Management Model of U.S. EPA was used and the actual pressurized flow routing through the sewer network was done by means of the EXTRAN model also developed under the EPA sponsorship. Both models have been described elsewhere (Huber et al., 1982; Roesner et al., 1982) and references to the model structures are here limited.

3.1 Computational Considerations

In the EXTRAN model, the sewer system is defined as a set of nodes (sewer junctions) which are connected by links (sewer pipes). The sewer system investigated was schematically represented by 21 nodes and 20 connecting links (see Fig. 3). The head losses at the 21 junction manholes belonged to the three types discussed in the earlier sections. In the entire system, there are eight straight-flow-through junctions, nine junction manholes with bends, and nine T-junctions with perpendicular laterals. The computational time step was selected as 20 seconds to maintain the stability of numerical procedures. Further details are given elsewhere (Marsalek 1984a).

The EXTRAN model used did not allow an explicit consideration of head losses at sewer junctions. Such losses, however, could be considered and simulated by increasing the downstream conduit roughness. In this case, the equivalent head loss, H_{eq}, can be expressed as

$$H_{eq} = H_j + H_c \qquad (6)$$

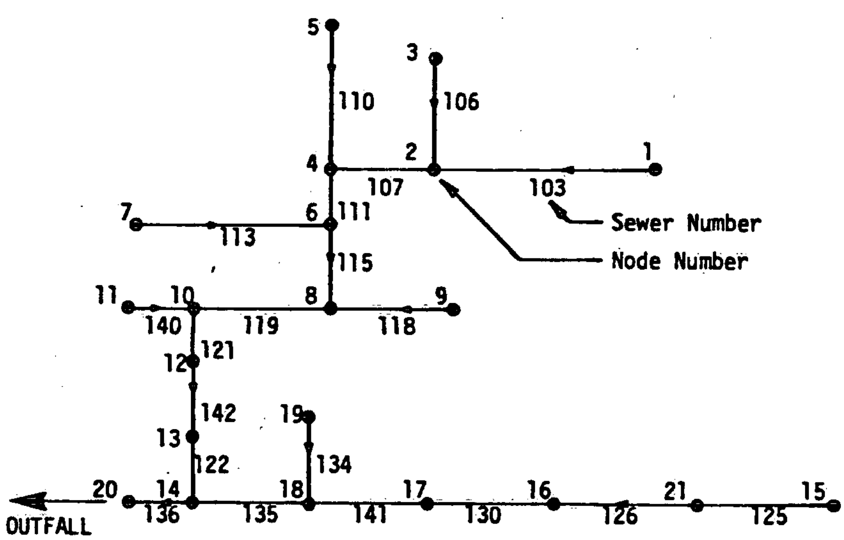

Figure 3 Schematized sewer system layout for pressurized flow routing

where H is the head loss, and subscripts j and c refer to the junction and conduit, respectively. After substituting for H_j (= K $v^2/2g$) and for H_c from the Manning's equation, the following expression is obtained:

$$H_{eq} = K \frac{v^2}{2g} + \frac{v^2 \, n^2 \, 4^{1.33}}{D^{1.33}} L \qquad (7)$$

where all parameters have been defined before, except the conduit length L.

It is possible now to define a conduit with the equivalent roughness which will have the same friction head loss as that given by Equation (8). Such a roughness is defined by the following expression:

$$n_{eq} = \left(n^2 + 0.008 \frac{K \, D^{1.33}}{L} \right)^{0.5} \qquad (8)$$

Equation (8) was used to calculate equivalent roughness for all conduits used in flow routing and these values are shown in Table IV. Assuming the conduit roughness as n = 0.013, the equivalent roughness values were calculated in the range from 0.0142 to 0.0171. Using the equivalent roughness values, runoff simulations were undertaken for a number of actual storms of various intensities. The simulation results are further discussed below with reference to the drainage system hydraulics.

3.2 Drainage System Operation

The pressurized flow routing reflects realistically the operation of the sewer system. For frequent storms, the transport of flow through sewers is in the form of the open channel flow. Such conditions are usually assumed in the initial design, although as mentioned before, every drainage system becomes sometimes surcharged when inflow exceed design conditions and no inflow controls are used. In the open channel flow, the minor losses at manholes are rather small and can be compensated for by relatively small invert drops at junctions. Once the design conditions are exceeded, the system starts to surcharge. The flow in some or all pipes changes from the free flow to the pressurized flow with an accompanying increase in flow velocities and the resulting increase in junction head losses. As the stormwater inflow further increases, the hydraulic grade line further rises, till it reaches the ground surface and stormwater flows out of the system at fully surcharged junctions. At this point, no more runoff can enter the sewer system and the water on the surface is transported by the major drainage system (i.e., road gutter, swales, etc.). As discussed below, the attainment of this state of complete system saturation is affected by the junction head losses.

In the earlier work on urban runoff peak frequencies (Marsalek, 1984a), a series of urban runoff simulations was done using both the SWMM and EXTRAN models. Some of the results are presented to further illustrate the operation of a surcharged drainage system. For this purpose, the flow routing results are discussed for storms Nos. 126, 144, 123, and 151, details of which were given elsewhere (Marsalek, 1984a).

For benched junctions, the envelopes of maximum hydraulic grade lines are plotted in Figure 4 for the longest route in the sewer system passing through junctions Nos. 1-2-4-6-8-10-12-13-14-20. Depending on the storm, the sewer system surcharges to various extent. For the least severe storm, this surcharge is relatively minor at the downstream end and somewhat increases in the upstream direction towards the junction No. 1. There was no

TABLE IV

CHARACTERISTICS OF THE SEWER NETWORK USED IN EXTRAN FLOW ROUTING

Pipe Number	D (m)	Length (m)	Slope	t_c (s)	Junction Head Loss Coeff.	Invert Drop (m)	Equivalent Manning's n
103	.458	209	.0051	98.6[3]	1.2	0.00	0.0146
106	.305[1]	91	.0132	52.6	1.2	0.19	0.0148
107	.458	91	.0132	42.9	1.4	0.11	0.0155
110	.381[1]	122	.0120	63.1	0.2	0.15	0.0141
111	.534	57	.0120	24.9	0.4	0.08	0.0148
113	.458	192	.0050	90.6	2.2	0.53	0.0151
115	.610	74	.0100	30.3	1.2	0.70	0.0162
118	.534	107	.0200	46.7	0.2	0.15	0.0142
119	.686	117	.0120	45.1	1.4	0.06	0.0146
140	.458[1]	54	.0120	25.5	1.2	0.49	0.0161
121	.686	57[2]	.0090	22.0	0.2	0.00	0.0118
142	.763	64[2]	.0050	23.4	0.2	0.00	0.0118
122	.763	81	.0050	29.6	1.4	0.24	0.0171
125	.458	125	.0156	59.0	1.6	0.22	0.0147
126	.458	125	.0156	59.0	0.8	0.22	0.0144
130	.686	92	.0024	35.5	0.2	0.00	0.0144
141	.686	134	.0024	51.7	1.2	0.00	0.0155
134	.305	89	.0236	51.5	0.2	1.71	0.0148
135	.686	85	.0042	32.8	0.6	0.15	0.0152
136	.839	62[2]	.0086	21.6	0.3	0.00	0.0141

1 A slightly larger diameter was used in RUNOFF block simulations
2 The pipe length was increased (roughness reduced) to increase t_c
3 $t_c = L/\sqrt{gD}$

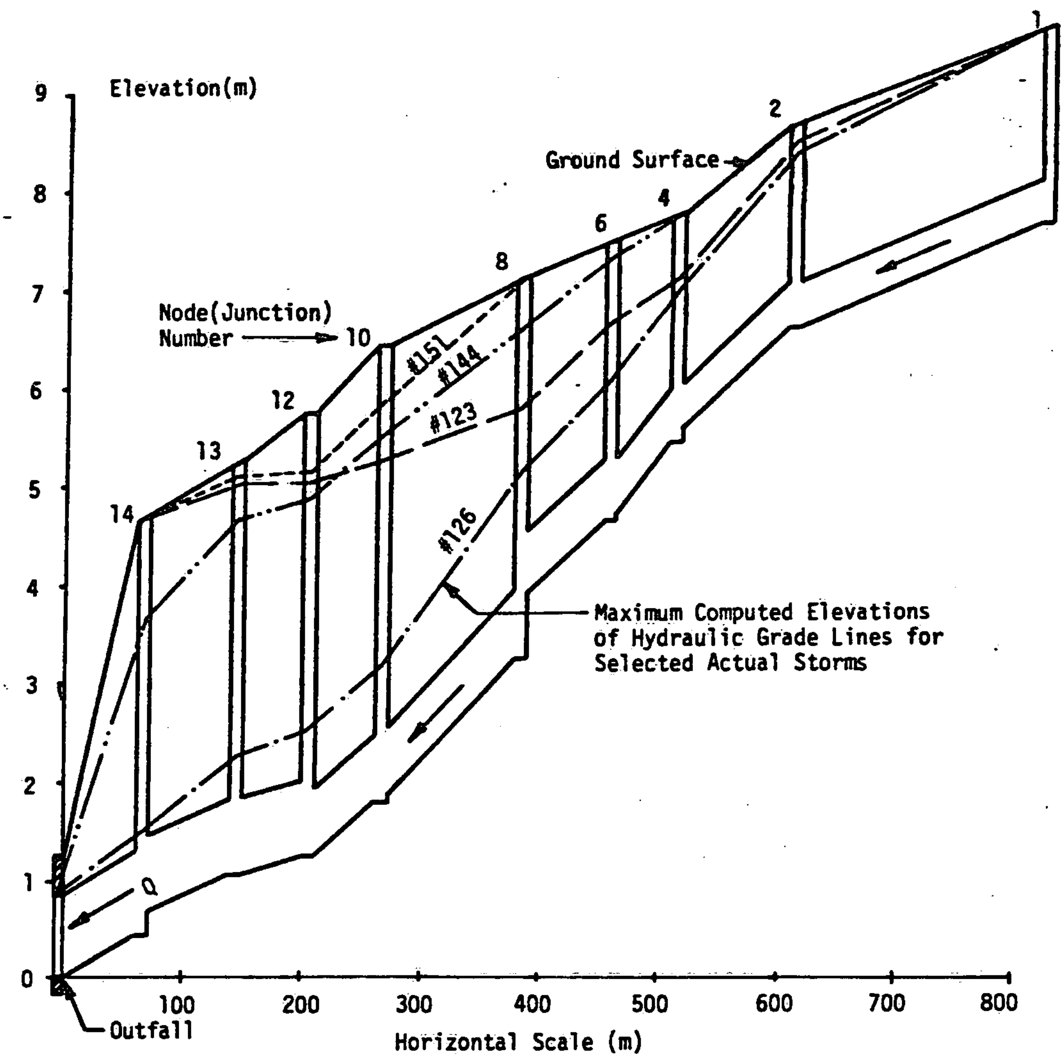

Figure 4 Hydraulic grade lines in a surcharged drainage system

appreciable outflow of stormwater at this junction, because the hydraulic grade line just about reached the ground surface at the junction. With the increasing storm severity, the surcharge and the elevation of the hydraulic grade line increase as well. Because of additional inflows along the discussed route, there may be flow reversals during some parts of the storm and the hydraulic grade line may temporarily rise in the downstream direction.

For the three most severe storms Nos. 144, 123 and 151, the node No. 1 was fully surcharged (i.e., to the ground surface) and this prevented additional inflow into the sewer system. For storm No. 123, the node No. 14 was also fully surcharged with the resulting outflow of stormwater and surface flooding. Finally, for the most severe storm No. 151, the junctions Nos. 1, 2, 4, 6, 8 and 14 were fully surcharged with the resulting outflow of stormwater.

It was further of interest to evaluate the contribution of junction head losses to the total head losses in the system. Such a contribution is difficult to demonstrate because of the dynamic behaviour of the drainage system and the system inflows varying in both time and space. For this reason, the friction and junction head losses were calculated and compared for individual conduits along the main flow route. The pertinent data are shown in Table V.

Table V shows results for two storms - the least and most severe. Other storms produced similar results. For individual pipes, the discharges at the time of the maximum outfall discharge are shown (Q's). In the next two columns, the junction head loss H_j and pipe friction head losses H_f are shown as used in the flow routing. Sums of junction head losses and friction losses are also shown. It can be inferred from the table that for benched systems, the junction head losses represent about 21% of all head losses along the flow route followed, but obviously depend on the characteristics of the sewer system. Table VI reflects the calculation for the same system without benchings in the manholes. Note the higher elevation the hydraulic grade line attains without benching.

TABLE V

TYPICAL HEAD LOSSES IN BENCHED SYSTEM

Pipe No.	Storm No. 126 Q (m^3/s)	H_j (m)	H_f (m)	Storm No. 151 Q (m^3/s)	H_j (m)	H_f (m)
103	.170	.065	.677	.178	.072	.742
107	.246	.159	.617	.249	.163	.632
111	.391	.062	.431	.289	.034	.235
115	.609	.266	.667	.362	.094	.236
119	.708	.262	.762	.801	.336	.976
121	.830	.051	.510	.985	.073	.719
142	.844	.035	.336	.980	.047	.453
122	1.030	.363	.633	1.430	.699	1.221
136	1.478	.109	.602	2.243	.252	1.385
TOTAL		1.374	5.235		1.769	6.599

TABLE VI

TYPICAL HEAD LOSSES IN NON-BENCHED SYSTEM

Pipe No.	Storm No. 126 H_j (m)	H_f (m)	Storm No. 151 H_j (m)	H_f (m)
103	.095	.677	.105	.742
107	.159	.617	.163	.632
111	.155	.431	.085	.235
115	.266	.667	.094	.236
119	.327	.762	.420	.976
121	.089	.510	.128	.719
142	.061	.336	.082	.453
122	.454	.633	.874	1.221
136	.218	.602	.420	1.385
	1.824	5.235	2.371	6.599

Junction head losses are largely controlled by the presence of benching. Neighbourhoods plagued with sewer backups during normal storms and served by nonbenched systems could possibly have the problem easily reduced to all but large storms by inserting benching.

If overflows are to be avoided, then benching also reduces the depth of excavation for the storm sewer pipe. That is, the same limiting capacity is obtained for less excavation.

3.3 Discussion

The consideration of junction head losses is worthwhile in the analysis of sewers flowing under pressure. The pressurized system studied conveyed significantly higher discharges (up to 74% higher, if the free outfall capacity is taken as 1.3 m^3/s) than the same system with the open channel flow. Thus there are some benefits to be derived from the pressurized system operation, but at the same time, steps must be taken to avoid flood damage. Such steps would involve for example inlet controls which would prevent excessive drainage surcharge.

For pressurized flow routing, it is preferable to work with the existing pressurized flow routing programs rather than to attempt to develop new ones. The EXTRAN model discussed here is highly suitable for this purpose, because of its widespread use and maintenance by various groups. Although the EXTRAN model does not consider explicitly junction head losses, the approach used here in the form of equivalent pipe roughness was found practical and eliminated the need to modify the original program. This approach, however, limits the use of some recent junction head loss data which may allow for variation in the surcharge depth and branch flow ratios. In particular, Lindvall's data (1984) indicate the possibility of head loss variations at straight-flow-through junctions with the surcharge. This cannot be accounted for by the equivalent roughness which remains constant throughout the flow simulation. Similarly, head losses at T-junctions vary with the flow ratio Q_2/Q_1 which may also vary during the runoff event and the head loss coefficient should be varied accordingly. It should be emphasized, however, that there is no evidence indicating that these additional refinements are justified for practical use, or that they can be fully supported by the existing experimental data on junction head losses.

4 CONCLUSIONS

A renewed interest in head losses at sewer pipe junctions led to a number of experimental studies which produced head loss data for a number of typical sewer junctions operating under a variety of conditions. In particular, the information on straight-flow-through junctions, junctions with a bend, and T-junctions is of a sufficient scope to cover typical design situations. Among various experimental factors, the alignment of branch pipes and the benching inside the junction manhole seems to be very important for junction design. Head loss reductions can be achieved by using full pipe depth benching at junctions and aligning branch pipes so that their axes intersect on the downstream head wall of two pipe junctions.

The information on junction head losses can be used advantageously in pressurized flow routing through sewer systems. The consideration of such losses produces more realistic results in terms of the hydraulic grade line elevations and reduced system capacity. Accurate computations of the hydraulic grade line elevations are important for considerations of flood damages and inconveniences arising from basement or surface flooding. For this purpose, the junction head losses can be accounted for by using the equivalent pipe roughness in the EXTRAN model. Further work on a pressurized flow routing model with an explicit consideration of junction head losses is recommended.

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