RRB-87-32

This manuscript has been prepared for the Canadian Journal of Civil Engineering and the contents are subject to change.

This copy is to provide information prior to publication.

HEAD LOSSES AT MANHOLES WITH A 90° BEND

by
J. Marsalek[1] and B.J. Greck[2]

[1]National Water Research Institute
River Research Branch
Canada Centre for Inland Waters
Burlington, Ontario, L7R 4A6

[2]Paul Theil Associates Ltd.
Brampton, Ontario, Canada
L6T 4V7

July 1987
NWRI Contribution #87-139

MANAGEMENT PERSPECTIVE

Canadian municipalities are facing enormous expenditures required to renovate their aging sewerage systems. In the renovation process as well as in new construction, it is desirable to improve sewer system capacities and collection efficiencies by proper design of sewer junctions. Consequently, the development of such design methods has been sponsored by more than 30 Canadian and U.S. municipalities and described in the report that follows. The report should be of primary interest to designers of municipal sewer systems and highway drainage.

PERSPECTIVE DE GESTION

Les municipalités canadiennes doivent dépenser des sommes énormes pour rénover leurs vieux réseaux d'égout. Lors de la rénovation de vieux égouts ou la construction de nouveaux, il est souhaitable d'accroîte la capacité des systèmes et l'efficacité de collecte au moyen d'une bonne conception des raccords. Des recherches sur les méthodes de conception ont donc été parrainées par plus de 30 municipalités canadiennes et américaines, recherches qui sont décrites dans le présent rapport. Ce rapport devrait avoir un intérêt capital pour les concepteurs de réseaux d'égout municipaux et d'ouvrages de drainage des routes.

HEAD LOSSES AT MANHOLES
WITH A 90° BEND

J. MARSALEK

Rivers Research Branch, National Water Research Institute

Burlington, Ontario, Canada L7R 4A6

(416) 336-4899

and

B.J. GRECK

Paul Theil Associates Limited

Brampton, Ontario, Canada L6T 4V7

(416) 792-2215

ABSTRACT

Head and pressure changes were studied at manholes with a 90° bend. For pressurized flow, such changes depended only on junction geometry. Among junction parameters, the benching was found particularly important. Full benchings reaching to the pipe crown produced the lowest head losses, particularly when combined with an enlarged pipe diameter at the junction. Head changes in open-channel flow were significantly smaller than those in pressurized flow.

Keywords: Head loss, manholes, sewer junctions, sewer design, sewer hydraulics.

RÉSUMÉ

La hauteur de refoulement et les changements de pression ont été étudiés dans des regards courbés à 90°. Dans le cas des écoulements sous pression, ces changements dépendaient uniquement de la géométrie du raccord. Parmi les paramètres de raccord, l'ancrage s'est révélé particulièrement important. Des ancrages entiers atteignant la couronne de la conduite permettent de réduire au minimum les pertes de charge, particulièrement lorsque le diamètre de la conduite s'élargit au raccord. Les changements dans la hauteur du refoulement dans les canalisations à ciel ouvert étaient considérablement moindres que ceux observés dans les canalisations sous pression.

Mots-vedettes: Perte de charge, regards, raccords d'égout, conception d'égouts, caractéristiques hydrauliques des égouts.

INTRODUCTION

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

While the friction losses have been extensively studied in the past and can be adequately characterized for practical purposes, only limited information is available on energy losses at sewer pipe junctions. Yet the losses at junctions may even exceed the friction losses and seriously limit the sewer system capacity. The sewer system may become surcharged and such conditions often lead to basement flooding or sewage overflows. Consequently, relief facilities may be required or new development halted in order to protect adjoining property. Such problems can be often avoided by minimizing form head losses in new as well as existing sewer systems.

Although the junction head losses as well as other form losses should be considered in the sewer design regardless of the design approach taken, the importance of such considerations has increased in recent years with the introduction of sophisticated computerized design methods. In the traditional sewer design based on hand calculations, sewer systems are designed as open-channel networks in which the form losses are not excessive and the hydraulic grade line does not exceed the pipe crown elevation. Under such circumstances even crude approximations of junction head losses may be adequate, particularly when dealing with subcritical flows of low velocities. There are, however, cases where sewer systems surcharge and increased head losses at junctions and hydraulic grade line elevations are of primary importance.

The surcharging of sewers occurs for various reasons. For example, in combined and storm sewers, surcharging is caused by the occurrence of rare storms which produce higher-than-design peak flows. During wet weather, surcharging may also occur in sanitary sewers because of high infiltration and inflow. Finally, it is sometimes economical to allow sewers to surcharge, to a limited extent, before any damages occur. Such a design is based on computerized pressure flow routing through the sewer network and on the calculation of the hydraulic grade line, which is maintained below the critical elevation above which flood damages occur. The accuracy and sophistication of such calculations is defeated by improper consideration, or neglect, of junction energy losses which can become fairly large in a surcharged system.

Recognizing the importance of junction energy losses in estimating the capacity and collection efficiency of sewer systems, the National Water Research Institute and the American Public Works Association (APWA) undertook a joint study of head losses at various types of sewer junction manholes. To conserve research funds, it was necessary to limit the study scope by establishing and following research priorities. Such priorities were established by APWA by surveying design practices of the study sponsors and municipal engineers. According to this survey, the highest priority was assigned to investigations of head losses at manholes with a 90° bend and to practical means of reducing such losses. Consequently, manholes with a 90° bend were studied in the first phase which is described in the paper that follows.

GENERAL CONSIDERATIONS

Previous Research

General problems of the flow through junctions have been discussed by Chow (1959) who concluded that the flow through junctions was a complicated problem whose generalization by analytical means was not possible and the best solution would be found through a model study of junction flow characteristics. Indeed, a literature search indicated that among various approaches, the study of head losses in physical models was the most common approach which was adopted for example by Sangster et al. (1959) and Hare (1983).

Sangster et al. (1959) studied junctions of a main pipe with a perpendicular lateral and, for a limiting case with no inflow through the main, produced some head loss data for manholes with a 90° bend.

Because of this limitation and the fact that only some of their data pertain to practical configurations, the applicability of their data is somewhat limited. Their experimental results indicate that the pressure changes increased with the increasing relative manhole width and the decreasing main pipe diameter, and were unaffected by the base shape.

Hare (1983) studied various layouts of manholes with a bend. In general, the configurations in which the inflow and outfall pipe axes intersected on the outfall junction wall produced the lowest losses. For a 90° bend, pressure changes decreased with the increasing outfall diameter.

Black and Piggott (1983) reanalyzed some of Sangster's and Hare's data and concluded that both the manhole layout and the relative inflow and outfall pipe sizes affected the pressure changes.

Following the literature survey, it was decided to study head losses at manholes with a 90° bend by experimental investigations of scale models of such installations. The selection of experimental variables was based on the dimensional analysis presented in the next section.

Dimensional Analysis

Considering the junction notation sketch in Fig. 1, the following customary definitions of junction head and pressure losses are introduced

[1] $$\Delta E = K \frac{V_o^2}{2g}$$

$$[2] \quad \Delta P = K_p \frac{V_o^2}{2g}$$

where ΔE is the energy head loss due to the junction, ΔP is the pressure head loss, K is the energy loss coefficient, K_p is the pressure loss coefficient, V_o is the mean velocity in the outfall pipe, and g is the acceleration due to gravity. In general, both coefficients K and K_p may attain negative values under special circumstances and, for that reason, they are sometimes also referred to as energy or pressure change coefficients. Coefficients K and K_p are not independent and for the manhole with a 90° bend the following relationship can be derived:

$$[3] \quad \Delta P + \frac{V_m^2}{2g} = \Delta E + \frac{V_o^2}{2g}$$

After substituting for ΔP and ΔE from eqs. (1) and (2), and $V_m = 4Q/\pi D_m^2$ and $V_o = 4Q/\pi D_o^2$, the final expression attains the form:

$$[4] \quad K_p = K + 1 - \left(\frac{D_o}{D_m}\right)^4$$

where V is the mean pipe velocity, D is the pipe diameter, Q is the discharge, and subscripts m and o refer to the main and outfall, respectively. For the common case of $D_m = D_o$, $K_p = K$.

For a steady pressurized flow through a particular junction design, which is further characterized by the junction benching, the head loss coefficient may be expressed as a function of 9 independent variables:

[5] $K = f_1 (\rho, \mu, Q, S, g, a, b, D_m, D_o)$

where f_1 is a function, ρ is the fluid density, μ is the fluid viscosity, S is the water depth at the junction, and a and b are the junction length and width, respectively. Dimensional analysis then yields the following expression for the head loss coefficient.

[6] $$K = f_2 \left(\frac{Q}{g^{1/2} D_o^{5/2}}, \frac{\rho Q}{\mu D_o}, \frac{S}{D_o}, \frac{a}{D_o}, \frac{b}{D_o}, \frac{D_m}{D_o}\right)$$

[7] $$K = f_3 \left(\frac{V_o}{\sqrt{g D_o}}, \frac{V_o D_o}{\nu}, \frac{S}{D_o}, \frac{a}{D_o}, \frac{b}{D_o}, \frac{D_m}{D_o}\right)$$

Among the independent variables, the first three are flow characteristics and the last three describe the junction geometry. Note that a similar expression would be obtained for the pressure loss coefficient. The flow characteristic parameters can be identified, from left to right, as the Froude number, the Reynolds number, and the relative junction submergence. For the conditions studied, eq. (7) may be further simplified. In particular, all experiments were done for a=b and D_m=D_o and the manhole geometry described by the base and benching shapes may be added to the right side of eq. (7)

[8] $$K = f_4 \left(\frac{V_o}{\sqrt{g D_o}}, \frac{V_o D_o}{\nu}, \frac{S}{D_o}, \frac{b}{D_o}, \text{base shape, benching}\right)$$

Further simplifications of eq. (8) are possible on the basis of the supporting data produced by the earlier researchers as well as in this study. It was shown by Sangster et al. (1959) that K does not depend on the Froude number in the region of pressurized flow and the Froude number can be omitted from further considerations. Black and Piggott (1983) reported that K is not affected by the Reynolds number for $Re > 10^4$. By keeping Re above this value in all runs, Re may be also omitted from the analysis. The effects of submergence were investigated in a special series in which the depth at the junction was varied by operating the control at the downstream end of the installation (Marsalek, 1985). The results of these investigations indicated that for S/D_o greater than 1.3 this parameter may be also omitted. Thus, the simplified form of eq. (8) indicates that for the conditions studied K depends only on the junction geometry:

$$[9] \quad K = f_8 \left(\frac{b}{D_o}, \text{base shape, benching}\right)$$

Model Similarity

Following investigations of head losses in scale models, it is of interest to establish whether the model results are directly transferable to the prototype. According to the theory of model similarity, the head and pressure change coefficients measured in the model should be directly transferable to the prototype if the independent parameters listed in eq. (9) are identical in both the model and the prototype. Such identities can be easily achieved in a geometrically similar scale model. Furthermore, this model should be operated for $Re>10^4$ and fair junction submergence ($S/D_o > 1.3$).

To verify the transferability of head or pressure change coefficients observed in scale models, a limited attempt was made to investigate possible scaling effects for square-base manholes with a 90° bend and three different benching designs. For this purpose, junction models of two sizes were built and tested for pressurized flow. The smaller junction was a 1:2 model of the larger junction. The detailed test results published elsewhere (Marsalek, 1985) indicate that the deviations of head loss coefficients observed in both installations varied from -10% to +11%, with a mean deviation of about 2.5%. Thus within the realm of experimental uncertainties, no significant scaling effects were found.

In summary, it appears that scale-model investigations offer the best approach to the study of head losses at surcharged sewer pipe junctions. Such models should be geometrically similar to the prototype junctions and operated in the region of $Re>10^4$ and $S/D_o > 1.3$. If these conditions are met, the head or pressure change coefficients observed in the model are directly transferable to the prototype.

EXPERIMENTAL INVESTIGATIONS

Experimental Factors

The dimensional analysis presented earlier indicated that head losses at manholes with a 90° bend are affected by manhole geometry which can be described by the manhole base shape, the relative manhole width, and the benching. The scope of variations of individual manhole parameters was based on the results of the earlier mentioned APWA survey of design practices. The individual manhole parameters are discussed below.

Regarding the manhole base shape, the APWA survey (Marsalek, 1985) indicated that the round-base manholes are predominant in the Canadian and U.S. practice. Square-base manholes, used by some Canadian municipalities or in the case of larger pipes, are less common. Regardless of the frequencies of use, both manhole types, with round and square bases, were studied.

The relative manhole size can be described by the ratio of the manhole characteristic cross-sectional dimension, either the diameter D_{mh} or the base width b, to the outlet pipe diameter. The frequency of use of various manhole and pipe sizes has been established from the APWA survey. Considering the sizes of model pipes and manholes, two basic D_{mh}/D_o (or b/D_o) ratios were obtained - 2.3 and 4.6. The smaller value corresponds closely to the maximum size sewer (0.61 m) installed in the most common standard manholes (D_{mh}=1.22 m). The larger value of D_{mh}/D_o (b/D_o) corresponded to the commonly specified minimum size storm sewers (0.25 to 0.31 m) installed in the standard 1.22 m manhole. It followed from the APWA survey that the range of D_{mh}/D_o (b/D_o) from 2.3 to 4.6 would cover at least two thirds of all design situations. The remaining design would be done mostly for larger non-prefabricated manholes whose relative sizes would be difficult to assess, but many of them would be still within the earlier mentioned range of relative manhole sizes.

In municipal design practice, it is customary to provide some benching at the junction manhole, in order to improve flow conditions at the junction and to reduce head losses. Five different benchings, referred to as designs B1-B5, were tested. Layouts of such benchings are shown in Fig. 2. A general description of individual designs follows.

Design B1 represented the simplest arrangement, in which no benching or flow guidance is provided at the junction. This design was expected to produce the greatest head loss and, although it is not very common in the current practice, it was included in the study as a limiting reference case.

Design B2 was obtained by extending the lower half of the pipe through the junction and adding horizontal benches extending from the semicircular channel to the junction wall. In plan view, the channel axis follows a 90° segment of a circle with a radius equal to one half of the manhole diameter (or base dimension). This type of benching is fairly common in municipal practice and it should generally result in lower head losses than design B1.

Design B3 represents an improved variation of design B2 obtained by extending the benching side walls vertically to the pipe crown elevation. It should provide even more flow guidance and hence lower head losses than B2.

Design B4 represents an improved variation of design B3. By rounding the pipe edges at the junction entrance and exit, further reductions in head losses should be possible. This design is not common in practice, although the square edges at both the pipe entry and exit are sometimes rounded, to some extent, in the field.

Design B5 was proposed to further reduce head losses by improving the junction hydraulic efficiency. It included the best features of design B3 and, by expanding the flow cross-section throughout the junction, the flow velocity and the corresponding head loss at the junction were further reduced. The changes in the pipe geometry were obtained by using eccentric pipe expanders/reducers of designs which can be implemented in the standard manufacturing process.

Although the above junction benching designs do not exhaust all the possible geometries, they represent a wide range of conditions from the worst case (B1) to the best practical case (B5). Experimental data obtained for these five designs can be used to make inferences for other designs.

Experimental Apparatus

The experimental apparatus used to study manholes with a 90° bend consisted of a water supply tank, the test pipes, the junction structure, and the outfall tank with a measuring weir.

Water discharging from a constant-head tank entered the water supply tank to which the main test pipe was connected. The water supply tank was designed to dampen out excessive turbulence in the flow discharging from the constant-head tank and to provide smooth inflow to the test pipe. The flow conditions in the main pipe were further improved by placing a honeycomb into the pipe inlet.

Two types of test pipes were used. The first one was a PVC pipe with an internal diameter of 152 mm. The second test pipe was a 75 mm clear acrylic pipe. Both test pipes consisted of individual sections which were connected by rubber-sleeve couplings. The main pipe upstream of the junction was 7 m long and the outfall pipe was 10.9 m long.

The test pipes were supported by a TV antenna beam resting on scissor jacks. These jacks were used to set the pipe slope. In pressurized flow tests, all pipes were set at a 1% slope. Piezometer openings were formed by drilling 3 mm diameter holes in test pipes at 0.5 to 1.0 m intervals. Typically, up to 28 piezometer openings were connected to a manometer board which allowed the reading of the piezometric heads with an accuracy of ±0.5 mm. To avoid possible errors in piezometer readings caused by pressure fluctuations during the reading, slides of the manometer board were taken and analyzed later.

The junction manhole was made of clear plexiglass. Two basic types were built - square- and circular-base manholes. The inside dimensions were 0.344 m x 0.344 m x 0.620 m (width x length x height) and the round-base manhole was formed by placing a sheet metal insert inside the square manhole.

At the downstream end of the outfall pipe, a special surcharge tank was built for controlling the level of surcharge. This tank had an adjustable gate, protruding from the bottom, at the outfall and by raising the gate, the level of surcharge in the test pipes was increased.

Water leaving the surcharge tank discharged into a tank with a 90° V-notch weir serving for flow measurement.

Experimental Procedures

In preparation for experimental runs, a selected manhole with an appropriate benching was installed. Flow through the facility was set in such a way that the Reynolds number and the level of surcharge would be sufficiently high. Once the flow through the installation was stabilized, piezometer readings were recorded at the manometer board photographically and the discharge was measured by the measuring weir. All the data were then processed by a computer program which calculated the total energy at individual points as $E = z + p/\gamma + V^2/2g$ (see Fig. 1). Finally, the energy grade lines upstream and downstream of the junction were individually approximated by least-squares straight lines fitted through the points measured. The difference between the upstream energy grade line and the downstream grade line, at the junction axis, was taken as the energy head loss. The same procedure was then repeated for various discharges.

The measured head losses were then plotted versus the velocity heads and the slope of the plot ΔE vs. $(V_o^2/2g)$ was taken as the head loss coefficient. A sample plot showing typical data scatter is shown in Fig. 3.

RESULTS AND DISCUSSION

The experimental program comprised 20 runs which were conducted for pressurized flow, square and round manhole bases, two relative manhole widths, five benching designs, and two model scales. Details and results of individual runs are listed in Table 1. The results were also plotted in Fig. 4. Further discussion follows.

The head losses in pressurized flow were investigated most extensively, because such conditions are of primary interest to sewer designers. In all experiments, significant head losses were observed at the junction. Although the pressure fluctuations in both pipe sections were relatively small, large agitation of the water surface at the junction manhole was observed. Under some conditions, particularly for design B2, a surface roller similar to that occurring in a hydraulic jump developed at the junction. Large eddies and vortices generated at the junction then affected the flow conditions in the downstream pipe. Such flow conditions were characterized by secondary helical currents. Effects of individual experimental factors are discussed below.

Manhole Base Shape

Within the range of experimental conditions studied, the effect of the manhole base shape was minimal. Considering all the paired results for square and circular bases in Table 1, for various D_{mh}/D_o's (b/D_o's) and benchings, the average head loss coefficients were 1.25 and 1.31, respectively. For individual benchings, the largest deviation was about 20%. Thus, although the round-base manholes produced head loss coefficients about 5% larger than those corresponding to the comparable square-base manholes, such a difference is hardly significant. Such results are consistent with the findings of Sangster _et al_. (1959) who simply grouped square and round manhole data for benching B1 together. Further verification that round-base manholes tend to produce slightly higher head losses than square-base manholes was indicated by Archer _et al_. (1978) for benching B3 and deflection angles of 30° and 60°.

Relative Manhole Size

The relative manhole size is described here by the ratio of the manhole diameter, or the base dimension of a square manhole, to the outlet pipe diameter D_o. Two values of this ratio reflecting the current design practice were used, 2.3 and 4.6.

The observed results were barely sensitive to the variation in the D_{mh}/D_o ratio. In general, the head loss increased with an increasing relative junction manhole size, because the smaller manholes were more effective in deflecting the flow into the outfall pipe. It seems plausible that the head losses would be more affected by the D_{mh}/D_o ratio in the region of the lower values of this ratio, as indicated by Sangster et al. (1959). In that study, the lowest head losses were observed in manholes which were only slightly wider (by 5%) than the pipe. Such losses were about 18% lower than those observed for the D_{mh}/D_o ratio of 1.9 which is comparable to the lower value employed in this study. It should be stated, however, that the $D_{mh}/D_o < 2.0$ values are of little interest in practical design, as indicated by the earlier mentioned APWA survey of design practices (Marsalek, 1985). Within the range of common D_{mh}/D_o values, $2 \leq D_{mh}/D_o \leq 6$, the head losses seem barely affected by the relative manhole size (width).

Manhole Benching

The manhole benching affected strongly the observed junction head losses. In fact, among the experimental factors studied, the benchings inside junctions had the most pronounced effect on the head loss. By providing flow guidance at the junction, some flow momentum is preserved and the head losses are reduced. The first four benching designs studied were designed to gradually improve the flow guidance at the junction. Taking design B1 as the reference, the common municipal designs B2 and B3 (see Fig. 2) reduced the head loss by 6%

and 38%, respectively. In other words, the largest head loss coefficient observed, 1.75 (B1), was reduced to 1.65 for B2, and to 1.1 for B3. The relatively small improvement brought about by design B2 was surprising. Detailed observations of the operation of this benching revealed that it deflected the incoming stream upwards and caused large agitation and head loss at the junction.

A further attempt to improve design B3 by rounding the exit and entrance pipe edges was counterproductive. The head loss for the resulting design B4 was even slighly larger than that for B3 but this minor deviation (0.04) was not statistically significant.

The final attempt to reduce the head loss at the junction by improved geometry is represented by design B5 which incorporates several beneficial features leading to reduced losses. Firstly, the flow guidance is provided at the junction by a benching extending above the test pipe crown elevation. Furthermore, the pipe cross section upstream of the junction is expanded and this results in two additional benefits - the effective relative manhole size (D_{mh}/D_o) is reduced and the flow velocity at the junction, in the channel provided, is also reduced. Both these features result in smaller losses. Design B5 reduced the original loss (B1) from 1.75 to 0.65 or by about 63%. Even when compared to the municipal designs B2 and B3, design B5 represents a significant improvement.

When examining design B5, the pipe transitions may seem to be relatively sharp and somewhat crude. Such a design, however, reflects the procedures used in manufacturing these transitions from sections of concrete pipes. These procedures impose constraints on further possible improvements in streamlining this transition. The application of design B5 in practice will be also affected by cost considerations. The transition sections required for B5 are generally hand-made and the associated increase in costs may limit its use. There will be, however, design situations where the need to reduce the head loss may call for the use of this design and override the cost considerations.

The last design tested was referred to as B1A. This design represents a minor variation of the reference benching B1. Such a variation was obtained by bringing the back junction wall (opposite the flow entrance) forward in order to align it with the exit pipe. This arrangement improves the energy recovery at the junction and slightly reduces the head loss (by 5%). The same idea could be implemented in conjunction with all the other benchings, but the head loss reduction is hardly significant.

Open-Channel Flow Results

Although the main emphasis was placed on head losses in pressurized flow, it was desirable to obtain some appreciation of the magnitude of such losses in subcritical open-channel flow. Towards this end, seven experimental runs were made and described in detail elsewhere (Marsalek, 1985). The results of such runs are summarized below.

Junction head losses in open-channel flow were significantly less consistent than those reported earlier for pressurized flow. The observed coefficients were always smaller than their counterparts corresponding to the pressurized flow. This was expected, because the changes in the flow cross-sectional area, encountered in the open-channel flow region at the junction, are smaller than those encountered in the pressurized flow experiments. The highest loss coefficient was observed for the reference design B1, 1.08. The loss coefficients for municipal designs B2 and B3 ranged from 0.27 (for B3) to 0.58 (for B2). The loss coefficient established for design B4 was exceptionally small, 0.07. It is believed that this value was strongly affected by the experimental data scatter. Finally, design B5 yielded a loss coefficient of 0.34. With the exception of design B4, the results found for the remaining four designs correspond quite well to the degree of changes in the flow areas at junctions with various benchings. It appears that manholes designed for pressure flow head losses should function quite well under open-channel flow conditions.

Sulphide Gas Releases

One of the concerns in sanitary sewer junction design indicated by the study sponsors is the release of sulphide gases caused by flow turbulence and agitation. Although such releases could not be directly studied in the experimental apparatus employed, some inferences about these releases can be made from visual observations of flow conditions at the junction. On the basis of such observations, benching designs B1 and B2 were found highly susceptible to possible releases of sulphide gases. Designs B3-B5 were rated as barely susceptible.

Design Data

For practical design, the head loss coefficients presented in Table 1 can be rounded off, extrapolated to the similar cases which were not studied, and the values of similar magnitude combined together. The recommended design data are listed in Table 2.

Head and pressure loss coefficients listed in Table 2 can be reduced by increasing the outfall diameter ($D_o > D_m$). Such changes can be accounted for by using an approximate correction proposed by Hare (1983) in the following form:

$$[10] \quad K_p\,(D_m/D_o) = C\,K_p\,(D_m/D_o = 1)$$

where C is the correction coefficient. C values are listed below for various D_m/D_o.

D_m/D_o	0.7	0.8	0.9	1.0
C	0.81	0.89	0.95	1.0

In this case, $D_m \neq D_o$, and the corrected head loss coefficient would be calculated by substituting K_p obtained from eq. (9) into eq. (4).

SUMMARY AND CONCLUSIONS

Head losses at manholes with a 90° bend are affected by both the flow velocity and the junction geometry. The loss increases linearly with the velocity head and the coefficient of proportionality is the head loss coefficient.

Among the junction geometrical parameters, the benching had the most pronounced effect on the head loss, followed by the relative manhole size (width), and the base shape. The lowest head losses were found for design B5 which represented a definite improvement in comparison to the municipal designs B2 and B3. Compared to the design without any benching (B1), the design with benching at half the pipe diameter (B2) brought about only an insignificant reduction in losses. The full benching at the pipe crown (B3), however, reduced the losses significantly. Observed head losses were barely sensitive to the variations in the relative manhole size tested. Smaller manholes produced somewhat smaller head losses, because they deflected better flow into the exit pipe. The losses observed for square- and round-base manholes were, on the average, almost identical.

Limited scaling tests with two models of different scales produced comparable head loss coefficients. No significant scaling effects can be deduced from the observed data.

Measurements of head losses in open-channel flow were less consistent than those conducted for pressurized flow. In general, the observed losses were always significantly smaller than those corresponding to the pressurized flow. The losses were again affected by the junction benching. Benchings B2-B5 significantly reduced the junction head loss.

REFERENCES

ARCHER, B., BETTES, F. and COLYER, P.J. 1978. Head losses and air entrainment at surcharged manholes. Hydraulics Research Station, Wallingford, U.K., report no. IT185.

BLACK, R.G. and PIGGOTT, T.L. 1983. Head losses at two pipe stormwater junction chambers. Proceedings, 2nd Nat. Conf. on Local Government Engineering, Brisbane, Australia, pp. 219-223.

CHOW, V.T. 1959. Open channel hydraulics. McGraw-Hill, New York, N.Y.

HARE, C.M. 1983. Magnitude of hydraulic losses at junctions in piped drainage systems. Civil Engineering Transactions, Institution of Civil Engineers, Australia, pp. 71-77.

MARSALEK, J. 1985. Head losses at selected sewer manholes. American Public Works Association, Chicago, Illinois, Special Report No. 52.

SANGSTER, W.M., WOOD, H.W., SMERDON, E.T. and BOSSY, H.G. 1959. Pressure changes at open junctions in conduits. ASCE Journal of the Hydraulics Division, 85(HY6), pp. 13-41.

ACKNOWLEDGEMENT

The authors would like to thank Richard Sullivan of the APWA for initiating and directing the study of manhole head losses and raising research funds from numerous sponsors. Special thanks are due to the study Steering Committee chaired by Tom McMullen of the City of Scarborough and all the study sponsors and advisors listed in the APWA report (Marsalek, 1985).

LIST OF SYMBOLS

a = the manhole length

b = the manhole width

B = the benching design

C = the correction coefficient accounting for differences in the main and outfall pipe sizes

D_m = the main (inflow) pipe diameter

D_{mh} = the diameter of a round-base manhole

D_o = the diameter of the outfall pipe

ΔE = the energy head loss (change) at the junction

g = the acceleration due to gravity

K = the head loss coefficient

K_p = the pressure change coefficient

μ = the fluid dynamic viscosity

ν = the fluid kinematic viscosity

ΔP = the pressure change at the junction

Q = the discharge

ρ = the fluid density

S = the depth of water at the junction

V_m = the mean flow velocity in the main pipe

V_o = the mean flow velocity in the outfall pipe

TABLES

Table 1. Manholes with a 90° Bend: Experimental Program and Results

Run No.	Manhole Base	D_{mh}/D_o b/D_o	Benching Design	$K=K_p$
1	SQ	2.3	B1	1.73
2	SQ	2.3	B2	1.46
3	SQ	2.3	B3	0.88
4	SQ	2.3	B4	0.93
5	SQ	2.3	B5	0.52
6	SQ	4.6	B1	1.52
7	SQ	4.6	B1A[1]	1.44
8	SQ	4.6	B2	1.69
9	SQ	4.6	B3	1.29
10	R	2.3	B1	1.80
11	R	2.3	B2	1.69
12	R	2.3	B3	0.97
13	R	2.3	B4	1.06
14	R	2.3	B5	0.55
15	R	4.6	B1	1.87
16	R	4.6	B2	1.67
17	R	4.6	B3	1.12
18[2]	SQ	2.3	B1	1.67
19[2]	SQ	2.3	B2	1.41
20[2]	SQ	2.3	B3	0.98

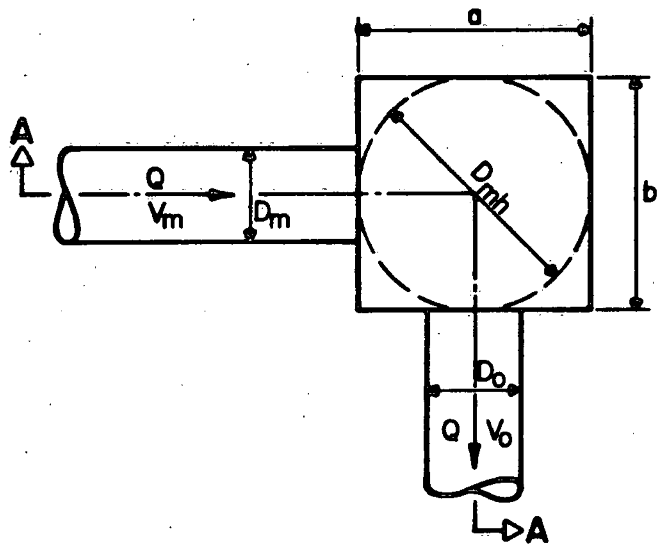
[1]Modified version of design B1.
[2]Scaling tests, the manhole dimensions were one half of those used in runs 1-3.

Table 2. Manholes with a 90° Bend: Design Data

Benching Design	Head (= Pressure) Change Coefficient[1]
	Smaller and Larger Manholes Combined ($D_{mh}/D_o = b/D_o$ = 2.3 - 4.6)
B1	1.75
B2	1.65
B3	1.10
B4	1.05
B5	0.65

[1]Pressure flow, no change in the pipe diameter at the junction.

FIGURE CAPTIONS



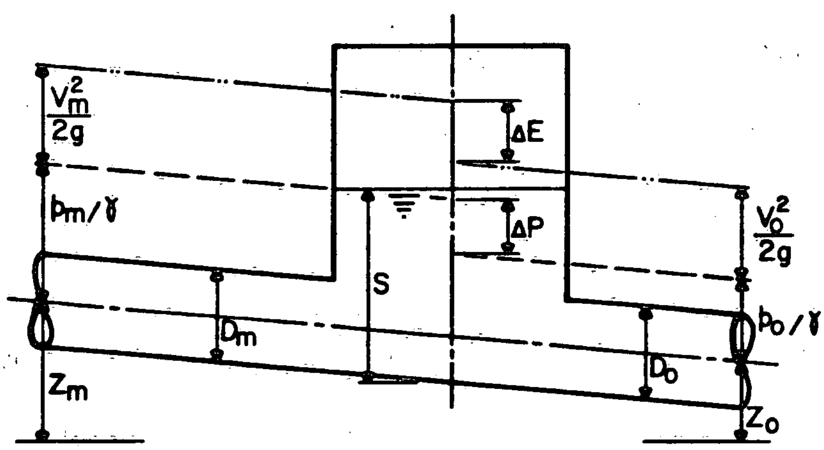


Fig. 1 Notation Sketch

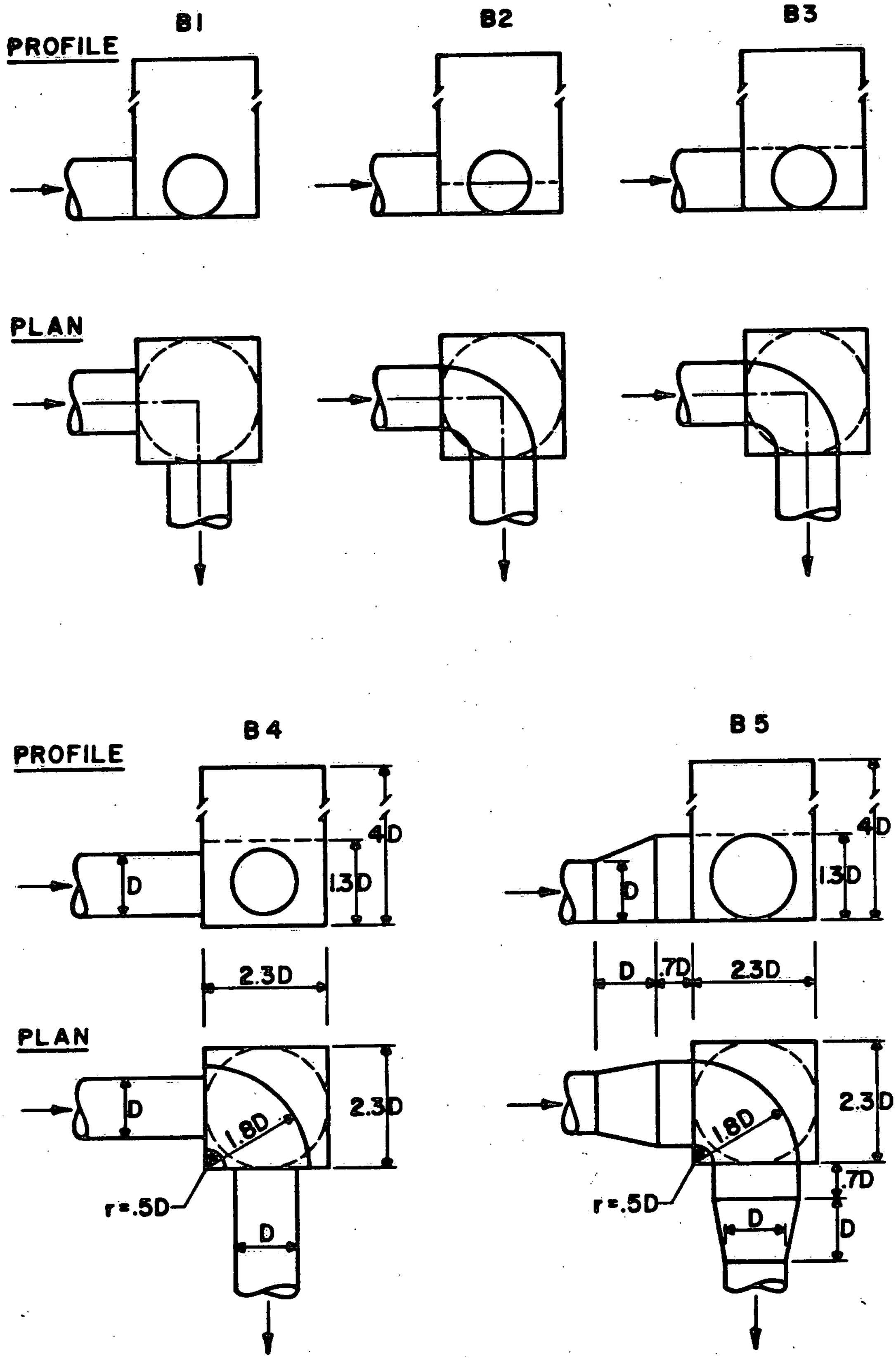


Fig. 2 Benching Designs Tested

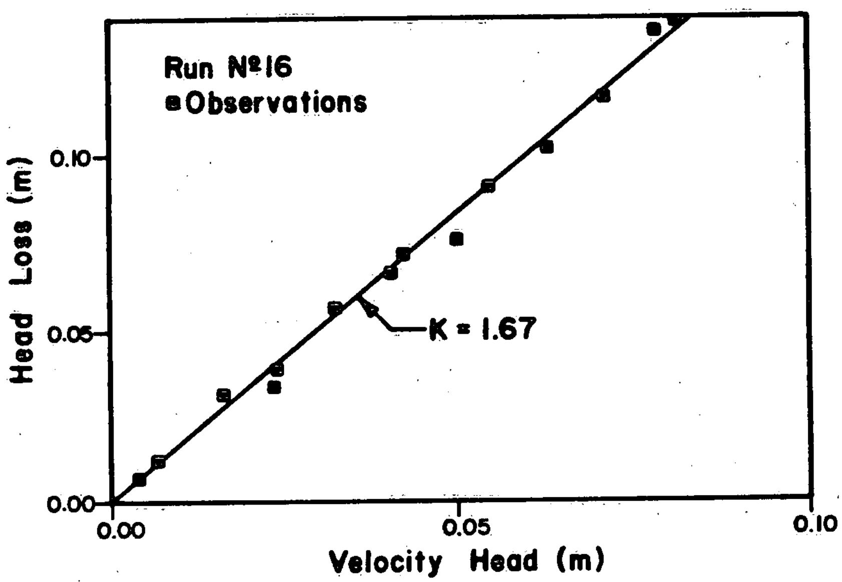


Fig. 3
Typical Plot of Head Loss vs. Velocity Head

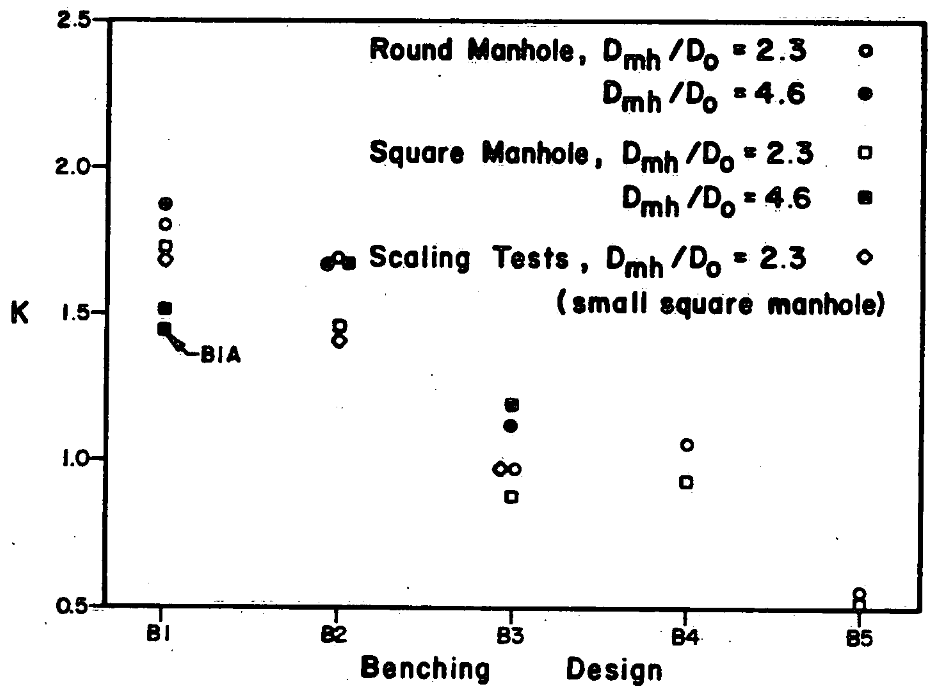


Fig. 4

Head Loss Coefficients for Manholes with 90° Bend