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SEWER JUNCTIONS

by

J. Marsalek

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RESEARCH PROGRAM FOR THE ABATEMENT OF MUNICIPAL POLLUTION WITHIN THE PROVISIONS OF THE CANADA-ONTARIO AGREEMENT ON GREAT LAKES WATER QUALITY

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ABSTRACT

Energy losses at manholes are not well defined. In this report energy losses at manholes where the pipe does not change direction have been measured in a scale model. Four different invert geometries were used with two plan forms but lateral inflows were not included.

Loss coefficients are given which may be used to improve hydraulic computations for sewer pipe networks.

SOMMAIRE

On connaît mal les pertes d'énergie subies aux regards d'égout. Dans le présent rapport, on a mesuré à l'aide d'un modèle réduit les pertes aux regards d'égout dans le cas où le tuyau ne change pas de direction. On s'est servi de quatre géométries inverses différentes et de deux coupes transversales, mais les débits entrants latéraux n'ont pas été inclus.

On donne des coefficients de perte qui peuvent servir à améliorer les calculs hydrauliques des réseaux d'égout.

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SUMMARY AND CONCLUSIONS

Energy losses at straight-flow-through sewer junctions with free-water surface were found to be considerably smaller than those corresponding to fully pressurized pipe junctions without the free-water surface. Typically, the losses at straight-flow-through sewer junctions amounted to only 0.05-0.35 of the velocity head.

In all cases, the mean energy losses observed for open-channel junctions (K=0.05-0.16) were smaller than those corresponding to the same junctions operating under pressure (K=0.10-0.35). Because of appreciable data scatter, the losses in the open-channel flow region were characterized by mean values, neglecting possible variations with the depth of flow and the pipe slope. For the manholes studied, there was a little difference between the losses observed for square and circular manholes. The losses were significantly affected by installing benching inside the manhole. Without any benching, the loss coefficient was about 0.15. This value was reduced down to 0.05 by installing a benching at the manhole.

For the pressurized flow, the junction energy loss coefficient increased about twice (K=.13-.35). Again, the coefficient was significantly affected by the benching inside the manhole. Without any benching, the loss coefficient varied from 0.20 to 0.35. With a benching extending to the pipe crown, the coefficient was reduced to 0.10-0.15.

Although the discussed junction losses are not excessive, they do reduce the overall system capacity, particularly in sewers with closely spaced manholes. Consequently, the junction energy losses should be compensated for by providing additional drops in the pipe invert. Such considerations are particularly important for surcharged sewer systems, in which flow velocities and the corresponding junction energy losses may become appreciable. Since the pressurized flow computations are typically computerized, a proper consideration of junction losses can be readily achieved using the energy loss coefficients presented in this report.

RECOMMENDATIONS

Energy losses at sewer junctions should be properly accounted for by using experimentally-derived energy loss coefficients. Such considerations are particularly important in the analysis of surcharged sewer systems where the elevations of the hydraulic grade line may be one of the design constrictions.

Experimental investigations of junctions with lateral inflows should be undertaken in the next phase of the study.

1 INTRODUCTION

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

While the skin 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 junction losses may often exceed the friction losses and seriously limit the sewer system capacity. Consequently, the sewer system may become surcharged and such conditions often lead to basement flooding or sewage overflows through overflow structures or onto the ground surface. A direct link between the junction losses and an increased incidence of combined sewer overflows was documented by the Borough of Scarborough which pioneered research in this area.

Although the junction losses (as well as other minor losses) need to be considered in the sewer design regardless of the design approach taken, the importance of such considerations increased in recent years with the introduction of sophisticated computerized methods. In the traditional piecemeal approach to sewer design based on hand calculations, even crude approximations of junction losses are acceptable because of large uncertainties involved and because such sewer systems are designed as open-channel systems (partly filled sewers). Consequently, the minor losses are not excessive and the hydraulic grade line does not exceed the pipe crown elevations.

Recent experience shows that significant savings can be achieved by allowing the sewer system to surcharge, to a limited extent, before any damages occur. Such a design is based on a computerized pressure flow routing through the sewer network and on the calculation of the hydraulic grade line which is maintained below the critical elevation to avoid flood damages. The accuracy and sophistication of such calculations is defeated by an improper consideration, or neglect, of junction energy losses which can become fairly large in a surcharged system.

Recognizing the importance of junction energy losses for establishing the capacity and collection efficiency of sewer systems, the Urban Drainage Subcommittee decided to commission the Hydraulics Division of the National Water Research Institute in Burlington to investigate energy losses at straight-flow-through junctions. The results of these investigations are presented in the report which follows.

PAST RESEARCH

A number of studies dealing with energy losses at sewer junctions was reported in the literature. The majority of these studies dealt with pressurized flow.

Chow (5) discussed open-channel junctions on the basis of works by Taylor (10) and Bowers (4). He concluded that the flow through a junction was such a complicated problem that its generalization was not possible and the best solution would be found through a model study of the flow characteristics involved.

Perhaps the most extensive tests of sewer junctions operating under pressurized flow conditions were undertaken by Sagster et al. (9) at University of Missouri. The junction performance was characterized by a pressure change coefficient K_2 defined as the change in the hydraulic grade line at the junction divided by the velocity head in the outflow pipe. For the straight-flow-through junctions with identical pipe sizes upstream and downstream of the junction, the pressure loss coefficient will approximate the energy loss coefficient. It was found that the coefficient K_2 did not depend on flow velocity, but only on the junction geometry. The coefficient K_2 increased with an increasing length of the junction box, but was not much affected by minor variations in the junction width. A round junction box yielded slightly higher values of K_2 than a square box of similar dimensions. For different pipe diameters upstream and downstream of the junction, the coefficient K_2 could be adequately described by conventional fluid mechanics equations.

Ackers (1) studied energy losses at junctions with and without changes in the pipe alignment at the junction. The observations were made in the field on a 0.1524 m (6") salt-glazed pipe for open-channel as well as pressurized flow conditions. Flow depth measurements were repeatedly made at two points - 0.915 m upstream and downstream of the junction. For manholes without changes in the pipe alignment, the energy loss coefficient varied from -0.13 for partly filled sewers to 0.16 for a slightly pressurized flow. No explanation was offered for energy gains - they probably resulted from limitations inherent in the experimental set-up. Considerably higher losses were observed for manholes with changes in the pipe alignment.

The most recent studies of physical models of sewer junctions include those by Jevjevich and Barnes (13), Townsend and Prins (11), and Archer, Bettes and Colyer (3). The first two studies dealt with a lateral inflow to the junction. Yevjevich and Barnes (13) studied a junction with a 90^{o} lateral pipe. The results were expressed as power losses which were approximated by an empirical equation. Townsend and Prins (11) studied most extensively a junction with a 45^{o} lateral. Apart from establishing energy losses at the junction, they also tested various structural means for reducing the losses.

Archer, Bettes and Colyer (3) investigated straight-flow-through junctions for pressurized flows. Both rectangular and circular junction manholes were studied. The loss coefficient remained constant for a wide range of velocities, but depended on the junction geometry.

In summary, model investigations offer the best approach to the study of sewer pipe junctions. Experimental energy losses were reported for a few selected junction arrangements. The majority of these observations were made for junctions operating under surcharge. Under those conditions, the energy loss appears to be proportional to the mean velocity head. The coefficient of proportionality is a constant given by the junction geometry.

3 STUDY SCOPE

The study reported here deals with straight-flow-through junctions of various geometries tested under a wide range of hydraulic conditions. For pipes running part-full, the bottom slope was of interest and was therefore varied in the following steps: 0.000, 0.001, 0.003, 0.006, and 0.010. For the first three slopes, the flow was subcritical. For the remaining two slopes, the slope was supercritical. For the pressurized flow, the pipe slope becomes unimportant. Surcharge heads up to three pipe diameters were obtained.

The junction geometry was varied in two ways - by considering two manhole shapes and four arrangements inside the manhole (see Figure 1).

Square and circular manholes were tested. The majority of tests were done for a fixed ratio of the characteristic manhole dimension to the pipe diameter. In a special test series, this ratio was slightly varied by reducing the manhole dimensions.

Special attention was paid to the arrangement inside the junction manhole. Altogether, four different arrangements, referred to here as moulds, were studied.

Mould No. 1 represents the simplest arrangement - no flow guidance is provided at the junction. The flow cross section expands suddenly at the entrance and contracts at the exit. This arrangement was expected to produce the largest energy losses.

Mould No. 2 provides some flow guidance through the junction by means of a square channel with the width and depth equal to the pipe diameter.

Mould No. 3 was obtained by extending the lower half of the pipe through the junction and by adding horizontal benches extending from the semicircular channel to the junction walls.

Mould No. 4 consists of the lower pipe half extended by vertical walls to the pipe crown elevation, where it connects to horizontal benches extending to the junction sidewalls. This arrangement was expected to produce the smallest energy losses.

Although the preceding four junction moulds do not exhaust all the possible geometries, they represent a fair range of conditions from the worst case (Mould No. 1) to the best practical case (Mould No. 4). Experimental data obtained for these four moulds can be used to make inferences for other junction geometries.

In all tests, the upstream and downstream pipe diameters were kept identical.

4 FUNDAMENTAL CONSIDERATIONS

The basic junction problem addressed here is limited to the straight-flow-through junctions without any changes in the pipe diameter or alignment. Two types of flow conditions at the junction are shown in Figure 2 for the open-channel flow and for pressurized flow.

In the open-channel flow region, the water surface at the junction exhibits some draw-down effects in the upstream half of the junction followed by a standing wave at the exit where the flow stream is impinging on the exit wall. The water surface profile at the junction is rather rough and surface disturbances propagate downstream of the junction.

Considering two sections upstream and downstream of the junction, the Bernoulli equation can be written in the following form:

$$z_1 + y_1 + \frac{\alpha_1 v_1^2}{2g} = z_2 + y_2 + \frac{\alpha_2 v_2^2}{2g} + \Delta E \qquad (1)$$

where z is the elevation, y is the depth of flow, v is the mean velocity, α is the kinetic energy coefficient, g is the gravitational acceleration, ΔE is the head loss between the two sections, and subscripts 1 and 2 refer to the upstream and downstream sections, respectively.

Assuming $\alpha_1 = \alpha_2 = 1$, the head loss ΔE can then be expressed as

$$\Delta E = (z_1 - z_2) + (y_1 - y_2) + \frac{(v_1^2 - v_2^2)}{2g} \qquad (2)$$

Head losses from all sources between sections 1 and 2 can be evaluated from experimental observations by means of Equation 2.

Minor losses attributed to the junction in general include the following: losses due to the flow deceleration upstream of the junction, losses at the entrance (a sudden expansion), losses due to turbulence inside the junction, losses at the exit (a sudden contraction), losses due to acceleration downstream of the junction, and losses due to surface waves and increased turbulence downstream of the junction. Because it is impossible to separate these individual losses, they are typically lumped together as junction energy loss which can be described by an energy loss coefficient. Such a loss is then considered as a sudden drop in the energy grade line, typically plotted at the centre of the junction. This approach was used here to analyze the observed data.

To apply Equation 2 to observed data, one would have to consider friction losses and to locate the upstream and downstream control section away from where there is a large energy grade line curvature (the vicinity of the junction). Such difficulties can be avoided by using the experimental data to establish energy grade lines well upstream and downstream of the junction and taking the difference between the two line elevations, at the junction axis, as the junction energy loss.

To design the experimental program the dimensional analysis of the junction problem was undertaken. For a particular junction configuration without lateral inflows, the energy loss can be written as

$$\frac{\Delta E}{D} = f_1 \left(S, \frac{v^2}{2gD}\right) \tag{3}$$

where D is the pipe diameter, S is the pipe invert slope, v is the mean velocity typically taken in the upstream pipe (for pressurized flow, the upstream and downstream velocities are identical), and g is the gravitational acceleration.

For pressurized flow, the invert slope becomes unimportant and Equation 3 reduces to

$$\frac{\Delta E}{D} = f_2 \left(\frac{v^2}{2gD}\right) \tag{4}$$

Thus, for a particular junction configuration, the dimensionless head loss appears to be a function of the pipe slope (i.e. for open-channel flow), and a dimensionless parameter similar to the Froude number.

Apart from the above experimental approach, attempts were made to evaluate the energy loss by applying the momentum principle to the junction problem. For the straight-flow-through junctions, the calculated losses exceeded substantially the observed ones and, consequently, this approach was abandoned.

5 EXPERIMENTAL APPARATUS AND PROCEDURES

5.1 Experimental Apparatus

A sketch of the experimetnal apparatus is shown in Figure 3. The apparatus consists of a water supply tank, the test pipe, 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 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 a smooth inflow to the test pipe.

The test pipe was a clear acrylic pipe 152 mm internal diameter. The pipe consisted of 14 sections each 1.83 m long. The sections were connected by flanges with rubber gaskets. The pipe section upstream of the junction was 16.47 m long, the downstream section was 9.15 m long.

The test pipe was supported by a TV antenna beam resting on 13 scissor jacks. The pipe slope was changed manually by gradually adjusting individual supporting jacks.

Piezometer openings were formed by drilling 3 mm diameter holes in the test pipe at 0.61 m intervals. Altogether, 40 piezometer openings were drilled and connected to a manometer board which allowed the piezometric heads to be read with an accuracy of $\pm$0.5 mm.

The junction manhole was made of clear plexiglass. Two basic types were built - a square manhole and a circular manhole. the inside dimensions of the basic square manhole were 0.344 m x 0.344 m x 0.620 m (width x length x height). A smaller manhole (0.241 x 0.241 x 0.620 m) was obtained by installing an insert inside the basic structure.

The basic circular manhole had an internal diameter of 0.293 m and the walls were 0.620 m high. An insert was used to reduce the manhole diameter to 0.203 m.

At the downstream end of the test pipe, a control valve was installed. Water leaving the pipe discharged into a weir box with a 90° V-notch measuring weir.

5.2 Experimental Procedures

The preparation for experimental runs comprised of the installation of a selected manhole with an internal mould and the adjustment of the pipe slope. The pipe slope was set approximately using a levelling instrument. The final adjustment was made using the piezometer readings along the pipe.

In experimental runs, the discharge through the apparatus was varied in small increments over a fairly wide range. For the lowest discharge, the depth of flow in the pipe was about 0.2 D. For high discharges, the pipe was surcharged and the hydraulic grade line was about 3 D above the pipe invert.

Once the flow through the installation was stabilized, piezometer readings were taken at the manometer board and the discharge was measured by means of the measuring weir. All these data were then processed by a computer program which calculated the total energy at individual points as

$$E = z + y + \frac{v^2}{2g} \tag{5}$$

where the notation is the same as used earlier and the kinetic energy coefficient α=1. Finally, the energy grade lines upstream and downstream of the junction were approximated by straight lines using two approaches. In the first one, the energy grade lines were determined as least-squares straight lines fitted through nine points upstream and nine points downstream of the junction. In both cases, the points were located between 1.72 m and 6.6 m from the junction. The second approach was somewhat analogous to that used by Archer, Bettes, and Colyer (3). The energy gradient was determined first for 20 piezometers upstream of the junction in the region virtually unaffected by the junction. Subsequently, mean energy line elevations were determined for seven points upstream and for seven points downstream (2.94-6.60 m from the junction) of the junction. The energy grade lines were then obtained by extending the earlier determined energy gradient through these mean elevations to the junction vertical axis. The difference between the two energy grade line intercepts with the axis was taken as the energy loss. The second approach was supposed to reduce possible effects of the energy line curvature, in the vicinity of the junction, on the losses calculated from the observed data. As discussed later, the differences between both procedures for determining experimental losses proved to be in most cases statistically insignificant.

Junction tests extended from the open-channel flow to the pressurized flow conditions. Typically, the measurements in the open-channel flow region exhibited a higher data scatter because of surface disturbances caused mostly by pipe deformations and by joints. For the proportional flow depth between 0.8 and 1.0, inherent flow instabilities were encountered and even small increases in the discharge caused the test pipe to surcharge.

The pressurized flow appeared to be more stable. In most experiments, the hydraulic grade line did not reach higher above the pipe invert than 3 D, because of limitations of the experimental apparatus. Such a range was found sufficient to determine the loss coefficient which apparently does not vary with the height of surcharging (3, 9).

Attempts to measure the depth of water at the junction were hindered by large agitation of the water surface. The depth was found to vary in both time and space.

6 EXPERIMENTAL RESULTS

6.1 Hydraulic Resistance of the Test Pipe

The hydraulic resistance of the test pipe was determined experimentally using the Darcy-Weisbach equation in the following form (7):

$$f = \frac{\Delta H}{L} \frac{2gD}{v^2} \tag{6}$$

where f is the Darcy-Weisbach friction factor, ΔH is the energy loss due to friction in a pipe section of length L.

Since all the quantities on the right-hand side of Equation 6 can be measured, the friction factor f can be determined from observations. Such a procedure was used in a series of experiments and the results are shown in Figure 4 as a plot of f vs. R, where R is the Reynolds number defined as $R=vD/\nu$ (ν is the kinematic viscosity). For comparison, the friction factors corresponding to hydraulically smooth pipes were calculated and also plotted in Figure 4.

The friction factor values calculated from observations were then used to determine the roughness factor k from the Colebrook-White Equation (2):

$$1/\sqrt{f} = -2 \log (k/3.7\,D + 2.51/R\sqrt{f}) \tag{7}$$

The best fit to experimental data was obtained for k=0.034 mm and the corresponding friction factors were also plotted in Figure 4. The relative pipe roughness was defined as $\varepsilon=k/D=0.034/152.4=0.000223$. Considering the relative pipe roughness and the range of R numbers from 1.08×10^5 to 2.49×10^5, it follows from the Moody diagram that the experiments were carried out in the transitional range where the friction resistance is controlled by both the pipe roughness and flow viscosity.

The best fit value of k=0.034 mm may appear somewhat large for the acrylic pipe. It would appear that the flange joints with rubber gaskets contributed to this increased value of k.

Finally, the Manning's roughness coefficient n, which is used commonly in the engineering practice, was also determined from the following equation:

$$n = r^{1/6} \sqrt{f/8g} \tag{8}$$

where r is the hydraulic radius. A mean value of n=0.087 was obtained and found to fit well within the range of values reported in the literature (2, 5).

6.2 Energy Losses at Junctions

The presentation of results is divided according to the type of flow, the type of manhole, and the internal manhole mould.

6.2.1 Losses in the open-channel flow region

Background - Open-channel flow through the junctions under consideration can be compared to a flow through a sudden expansion followed shortly by a sudden contraction. If one considers a long reach of a part-full sewer pipe with a junction, the water profile in this reach will approach the normal flow profile at some points upstream as well as downstream of the junction. Over the whole reach encompassed, the total energy loss for the case with the junction is the same as that for uniform flow without the junction. Thus, the energy head loss at the junction is a localized phenomenon which is compensated for by lower-than-normal flow losses upstream and downstream of the junction. Observations presented here and elsewhere (5) indicated that energy grade lines observed within 50 pipe diameters upstream and downstream of the junction yielded local junction head losses.

Energy loss measurements were made in the open-channel flow region for two types of manholes fitted with internal moulds and for five various pipe slopes. As mentioned earlier, an appreciable experimental scatter was experienced mostly because of water surface disturbances which affected the measured flow depths and the calculated velocity heads. Such disturbances are then reflected, with an amplified amplitude, in the calculated energy grade line. Inherent flow instabilities hindered the measurements for depths larger than 0.8 D. In spite of these limitations, the experimental energy head losses presented below offer good guidance for estimating junction losses in partly-filled sewer pipes.

Flow Characteristics - The flow depth typically varied from 0.2 D to 1.0 D. By varying the pipe slope, both subcritical and supercritical flow regimes were obtained. Subcritical flows were typically obtained for invert slopes from 0.000 to 0.003, for steeper slopes, the flow became supercritical with Froude numbers as high as 2.88. The flow regimes upstream and downstream of the junction were identical. However, in four runs, the subcritical flow (Fr=0.93) upstream of the junction changed to a supercritical flow (Fr=1.05) downstream of the junction.

Reynolds numbers were determined for individual runs and were found to vary from 0.48×10^4 to 3.07×10^5.

<u>Junctions Tested</u> - Energy losses were observed for both square and circular manholes with four internal moulds. The square manhole had a base 0.344 x 0.344 m, thus yielding a ratio $D_{pipe}/a=0.1524/0.344=0.443$. The circular manhole was 0.293 m in diameter, thus yielding a ratio $D_{pipe}/D_{manhole}$ $=0.1524/0.293=0.520$. Both manholes were investigated with the four internal moulds described earlier (see Figure 1).

<u>Results</u> - Experimental losses are plotted for square and circular manholes in Figure 5 and 6, respectively. In each figure, the dimensionless losses $\Delta E/D$ were plotted vs. $v^2/2gD$ for various moulds and invert slopes.

The data plotted in Figures 5 and 6 confirm the findings of the dimensional analysis that the dimensionless head loss depends on the dimensionless velocity head $v^2/2gD$. Because no significant effects of the invert slope on the observed head losses were apparent from the experimental data, the data for all slopes were grouped together and approximated by straight lines which could be described by the following equation:

$$\frac{\Delta E}{D} = K \frac{v^2}{2gD} \tag{9}$$

Thus, for individual junction configurations, the energy head loss is linearly proportional to the velocity head. The coefficient of proportionality is a constant given by the junction geometry. This constant is referred to as the head loss coefficient.

The effect of the junction geometry, described here as moulds M1-M4, on the observed head losses was quite appararent. The larger the change in the flow cross section at the junction, the larger the energy head loss. The largest changes in the flow cross section occur for mould M1, the smallest changes correpond to mould M4. The same ranking applies to the observed head losses. Moulds M2 and M3 are quite comparable.

The energy head loss coefficients found by fitting Equation 9 to observed data are listed in Table 1. It appears that the local head losses in the open-channel flow junctions represent only a small fraction of the velocity head.

TABLE 1. MEAN VALUES OF THE ENERGY LOSS COEFFICIENT FOR OPEN-CHANNEL FLOW CONDITIONS

Manhole Type	Energy Loss Coefficient K			
	Mould			
	M1[1]	M2[1]	M3[2]	M4[2]
Square	.149	.094	.081	.043
Circular	.141	.088	.112	.046

[1] Mean values for d=0 - 1.0 D

[2] Mean values for d=0.5 D - 1.0 D (for d<0.5 D, K=0)

6.2.2 Losses in the Pressurized Flow Region

Background - Tests of junctions were extended to the pressurized flow region. The experimental apparatus allowed the hydraulic grade line to rise up to 3 D above the pipe invert. Since the earlier studies indicated that the loss coefficient remained constant in the pressurized flow region regardless of the flow velocity, even a somewhat limited range of velocities, available in this study was sufficient to establish the loss coefficient value.

For the pressurized flow, the pipe slope becomes unimportant and, therefore, all the results for various slopes were grouped together.

Both square and circular were tested. The basic manholes had dimensions of the square and circular base 0.344x0.344 m and D=0.293 m, respectively. Manholes of other sizes were tested on a limited scale, as described below.

Results - Experimental energy head losses were plotted as $\Delta E/D$ vs. $Kv^2/2gD$ in Figure 7 and 8 for square and circular manholes, respectively. For brevity, only the losses obtained by fitting straight lines to upstream and downstream energy heads (i.e. the second method, see Section 5.2) are shown in these figures.

For individual moulds, head loss coefficients were obtained by fitting straight lines, passing through the origin, to the plotted data. The head loss coefficients obtained this way are listed in Table 2 for two methods of calculating head losses from observed data. A comparison of the two methods shows a little difference. Consequently, only the coefficients calculated by the second method, which is more common, are recommended for further use.

TABLE 2. JUNCTION ENERGY LOSS COEFFICIENT FOR PRESSURIZED PIPE FLOW

	Energy Loss Coefficient Mould							
	M1		M2		M3		M4	
Manhole Type	K_1[1]	K_2[2]	K_1	K_2	K_1	K_2	K_1	K_2
Square	.334	.349	.179	.203	.216	.203	.154	.123
Circular	.210	.208	.109	.118	.143	.175	.110	.117

[1] ΔE calculated from mean energy grade elevations upstream and downstream and the fitted gradient.

[2] ΔE calculated from fitted energy grade lines upstream and downstream

Energy losses at pressurized flow junctions were appreciably larger than those described earlier for the open-channel flow conditions. This was primarily caused by larger changes in the flow cross section at the pressurized junction.

The effect of the junction moulds on energy losses was quite obvious. The largest losses were observed for mould M1, followed by moulds M3, M2, and M4. This ranking is quite similar to that presented earlier for the open-channel flow.

The effect of the junction width on energy losses was investigated for mould M1 in a special series of experiments. In particular, the square manhole width was reduced to 0.241 m and later to 0.1524 m. As expected, the loss coefficient also descreased with the decreasing junction width. A similar finding was made for a reduced circular manhole 0.203 m in diameter. The results are listed in Table 3.

TABLE 3. EFFECTS OF JUNCTION WIDTH ON ENERGY LOSS COEFFICIENT (MOULD M1)

	Square Manhole			Circular Manhole	
D_p/a (D_p/D_m)	.443	.632	1.000	.520	.751
K	.349	.273	.203	.208	.143

The data shown in Table 3 follow the trend reported earlier by Sangster et al. (9) - the losses increase with the junction width. According to ref. (9), there were hardly any changes in the loss coefficient once the junction width increased past a certain limit - about 2.4 D (i.e. D/a=0.415). Such a width roughly corresponds to the maximum widths used in this study.

7 DISCUSSION OF RESULTS

Obtained experimental results indicate that the energy losses at straight-flow-through junctions with a free-water surface are considerably smaller than the values adopted from experiments for a fully-pressurized pipe flow without any free surface at the junction. The latter losses would typically vary from 0.8 to 1.5 times the velocity head. The lower value was calculated for mould M1 ($A_{pipe}/A_{junction}$=0.35) after reference (8):

sudden expansion	$K_e = 0.41$
sudden contraction	$K_c = 0.43$
combined loss	$K = 0.84$

The upper value (1.5) corresponds to K_e=0.50 and K_c=1.0 as proposed by Wood (12).

It appears that at sewer junctions, the main body of the stream crossing the junction remains more or less intact throughout the junction. Only the outlying parts undergo some changes in the trajectory and these changes then contribute to energy losses. The overall energy loss remains, however, fairly small.

It is of interest to compare the values of the energy loss coefficient K reported here to those rpeorted elsewhere. Such a comparison is presented in Table 4.

TABLE 4. STRAIGHT-FLOW-THROUGH JUNCTION ENERGY LOSS COEFFICIENTS FROM VARIOUS SOURCES

Source	Manhole Type	D_p/a (D_p/D_m)	Benching at Junction	Energy Loss Coefficient K	
				Open-Channel Flow	Pressurized Pipe Flow
Ref. 9	Circular	.303	None (M1)	---	.22 - .28[1]
	Square	.95 - .303	None (M1)	---	.05 - .20[1]
Ref. 3	Circular	Not known	M3	---	.15
	Square		M3	---	.10
This Report	Circular	.520	M1	.14	.21
	Square	.443	M1	.15	.35
	Circular	.520	M3	.11	.17
	Square	.443	M3	.08	.20

[1] Pressure change coefficient

In general, a fair agreement was found among data from various sources. The losses in Table 4 seem to vary from 0.08 to 0.35. The smallest losses correspond to the open-channel flow and some benching at the junction. Because the head losses in open-channel flow are fairly small and of a local character (i.e. they do not propagate upstream through the sewer system as in the pressurized flow), they may be neglected in practical calculations. The highest head losses in Table 4 correspond to pressurized flow and no benching.

For pressurized flow, the significance of sewer junction head losses depends on the spacing of manholes. To obtain some indication of the effects of junction head losses on sewer pipe capacity, the results of a sample calculation are presented in Figure 9. The calculation was made for a concrete pipe (D=0.915 m, L=300 m, f=0.024) assuming that the hydraulic gradients for pipes with and without junctions are the same. Considering practical manhole spacings (6) from 30 m to 100 m, the pipe discharge reduction, due to the junction head losses, would vary from 18 percent to 7 percent in the worst case (mould M1). Discharge reductions for close manhole spacings are significant and, consequently, the junction head losses should be considered in such cases.

A significant reduction in the energy head losses can be achieved by improving the junction geometry - by installing a benching at the junction. Such an arrangement reduces the flow cross-sectional changes at the junction. Compared to the junction without any benching (mould M1), significant reduction in energy losses were obtained by providing a channel conveying the flow through the junction. The magnitude of these reductions still depends on the geometry of this channel. The best results were obtained for a channel cross section formed by the lower half of the pipe and extended by vertical walls to the pipe crown elevation (mould M4). The next best arrangement was referred to as mould M2 - a square channel aligned with the sewer pipe and connected by horizontal benches to the junction sidewalls. From the practical point of view, these two arrangements, M2 and M4, may hinder other functions of the manhole, such as easy accessibility for maintenance purposes. Mould M3 (i.e. the lower pipe half with horizontal benches) may offer a compromise between an easy accessibility and reduced energy losses at the junction.

It is virtually impossible to investigate all possible geometries of sewer junctions. Consequently, the designer has to select the data for the type best approximating the actual design situation. It is felt that moulds M1 and M4 represent the worst and the best practical cases, respectively. Many other configurations will fall somewhere between these two extremes and their loss coefficients can be interpolated accordingly.

Effects of the pipe deflection, at the junction, have not been studied here. The literature data (3) indicate that, for a 30° deflection, the loss coefficient increased from 0.10-0.15 to 0.4-0.5, and reached values of 0.85-0.95 for a 60° deflection. For even larger deflections, the upper limit value of 1.5 (12) may be used.

It should be stressed that the minor losses discussed here (straight-flow-through junctions) are among the smallest minor losses encountered in sewer systems. Other types of losses, such as at junctions with laterals and pipe deflections, will be much larger and cannot be neglected. It is quite likely that in many sewer systems the minor losses caused by junctions, sewer inlets, house connections and other appurtenances will exceed the friction losses and will reduce significantly the system capacity.

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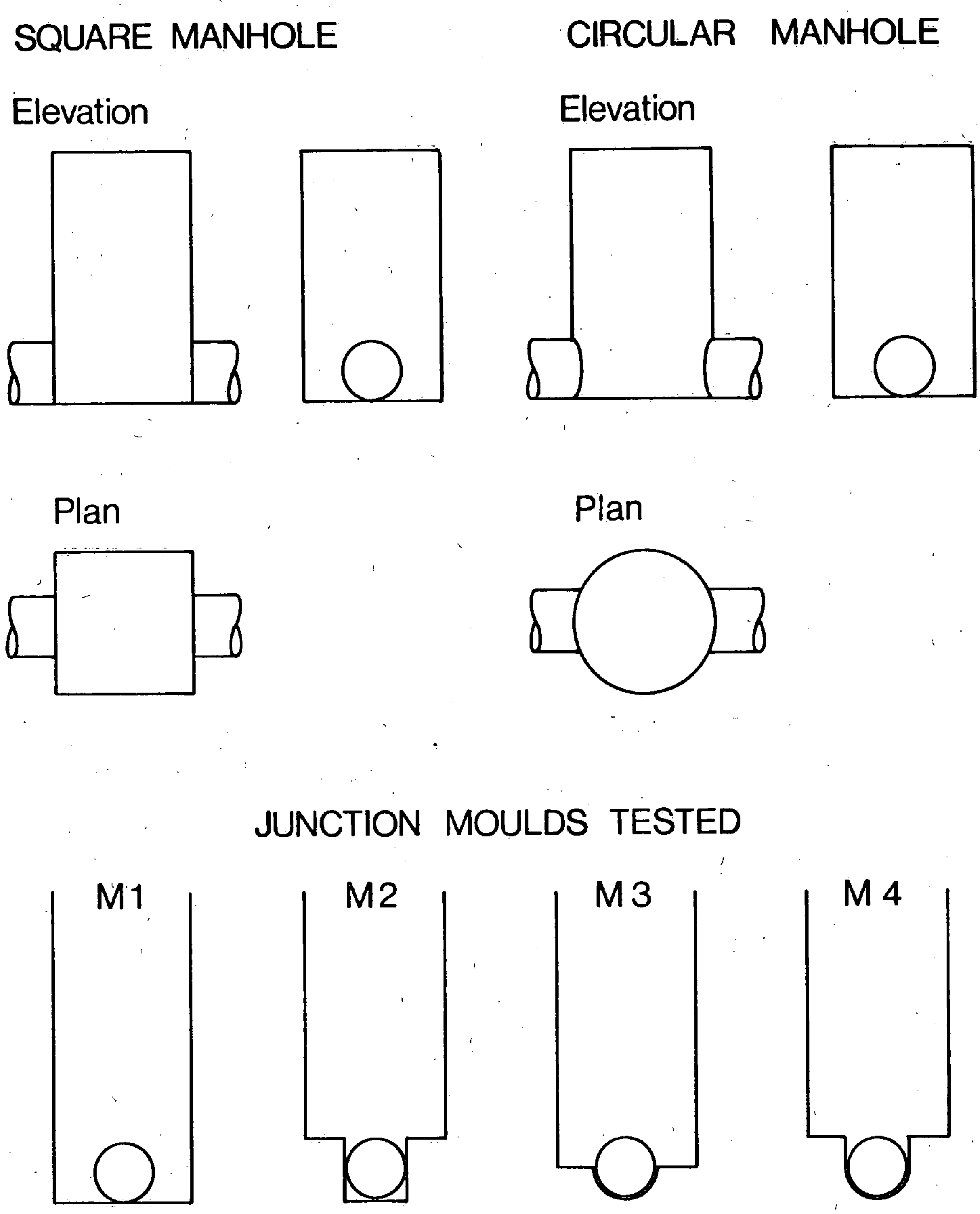
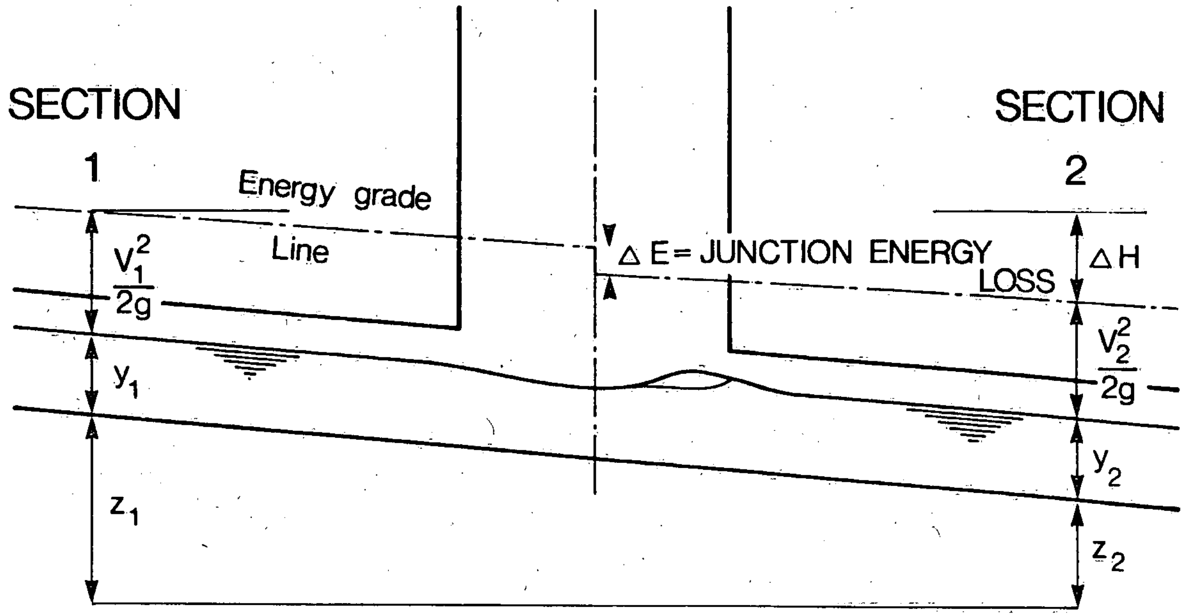
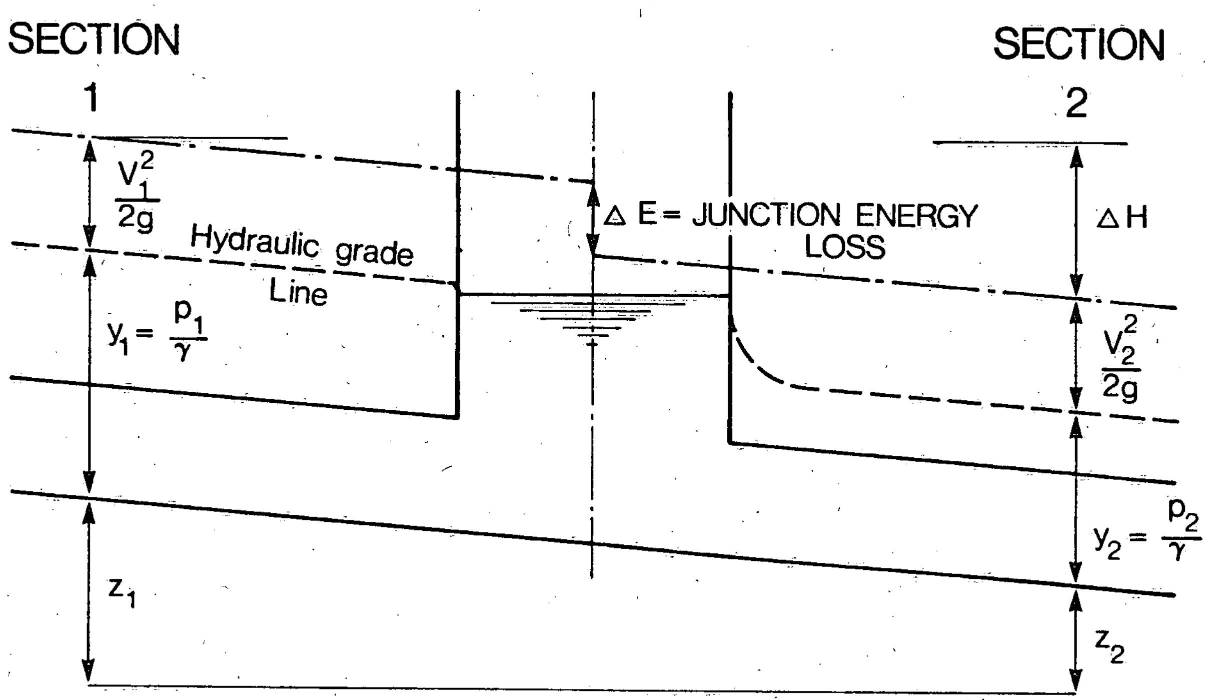


FIG. 1 SEWER PIPE JUNCTIONS TESTED



(a) OPEN – CHANNEL FLOW



(b) PRESSURIZED FLOW

FIG. 2 NOTATION SKETCH

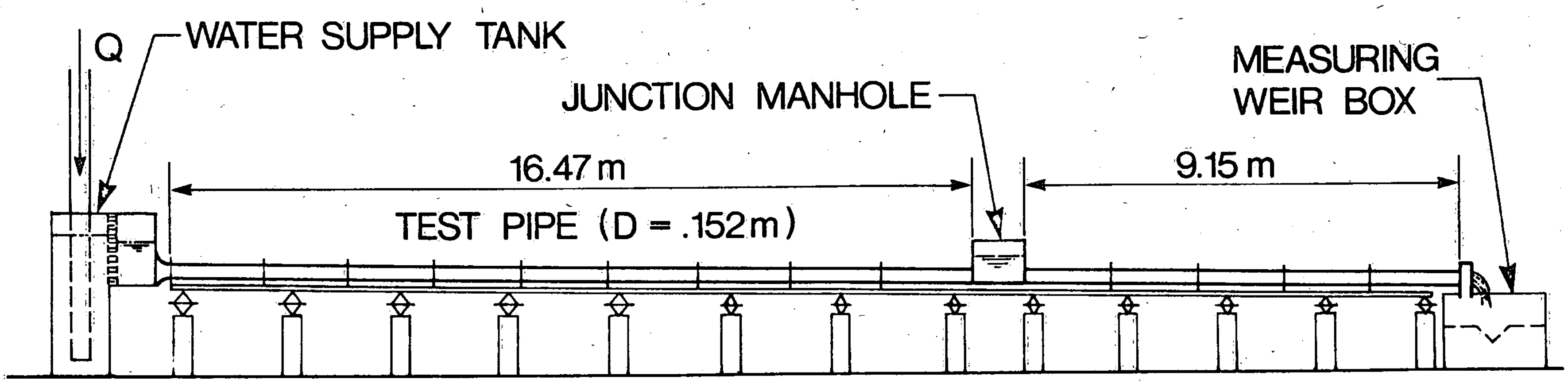


FIG. 3 EXPERIMENTAL APPARATUS

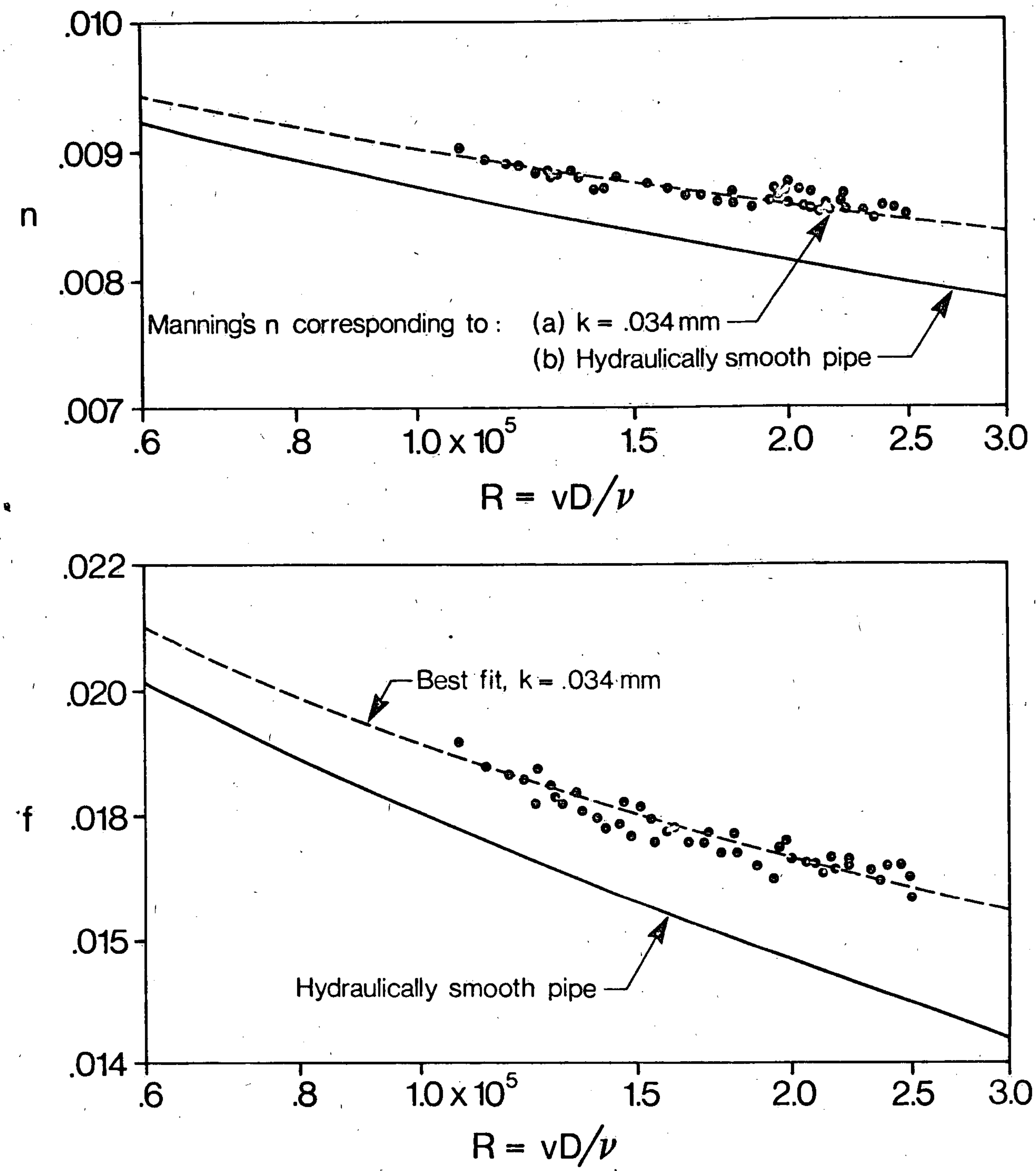


FIG. 4 FRICTION FACTOR OF THE TEST PIPE.

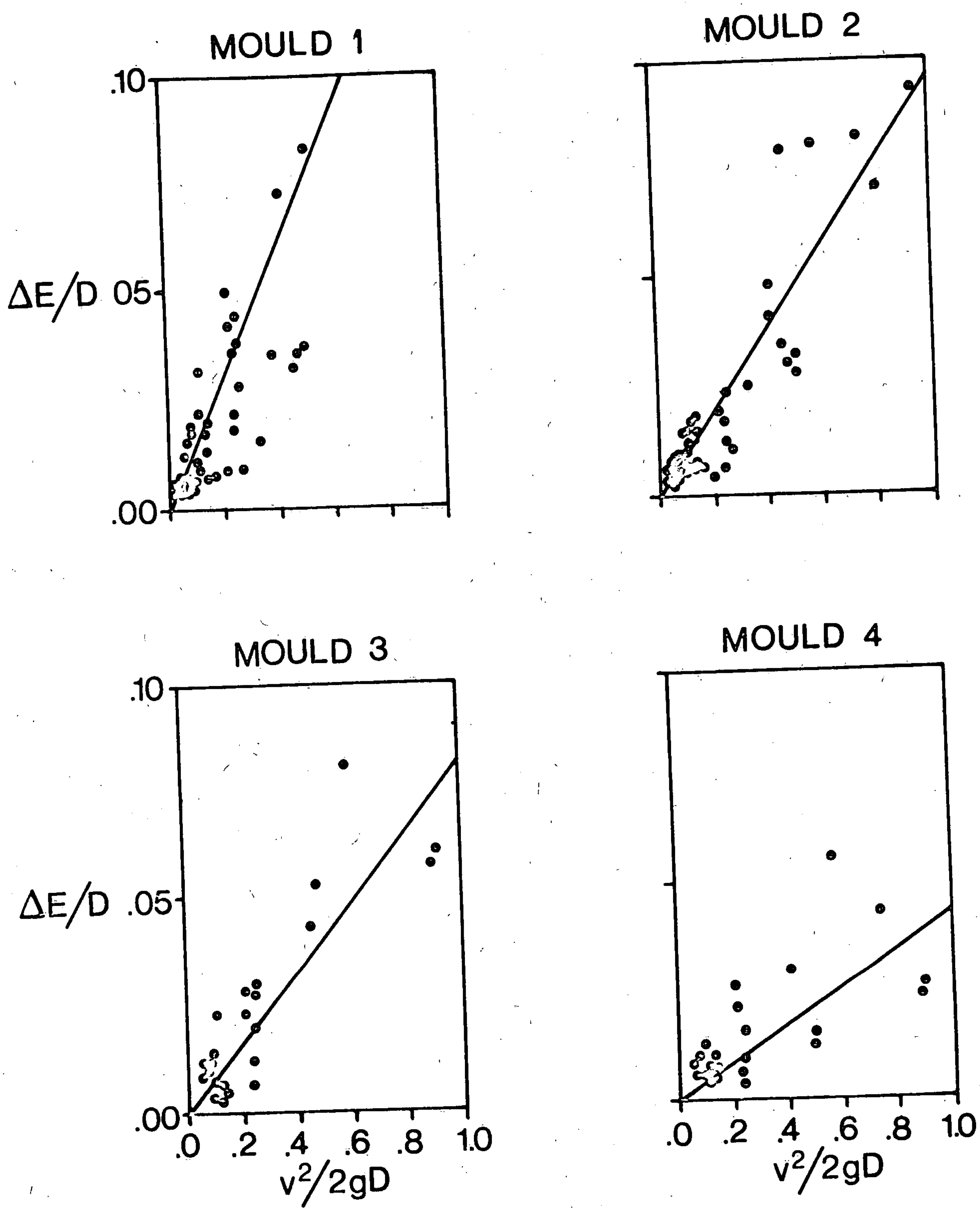


FIG.5 ENERGY LOSSES : SQUARE MANHOLE, OPEN-CHANNEL JUNCTION

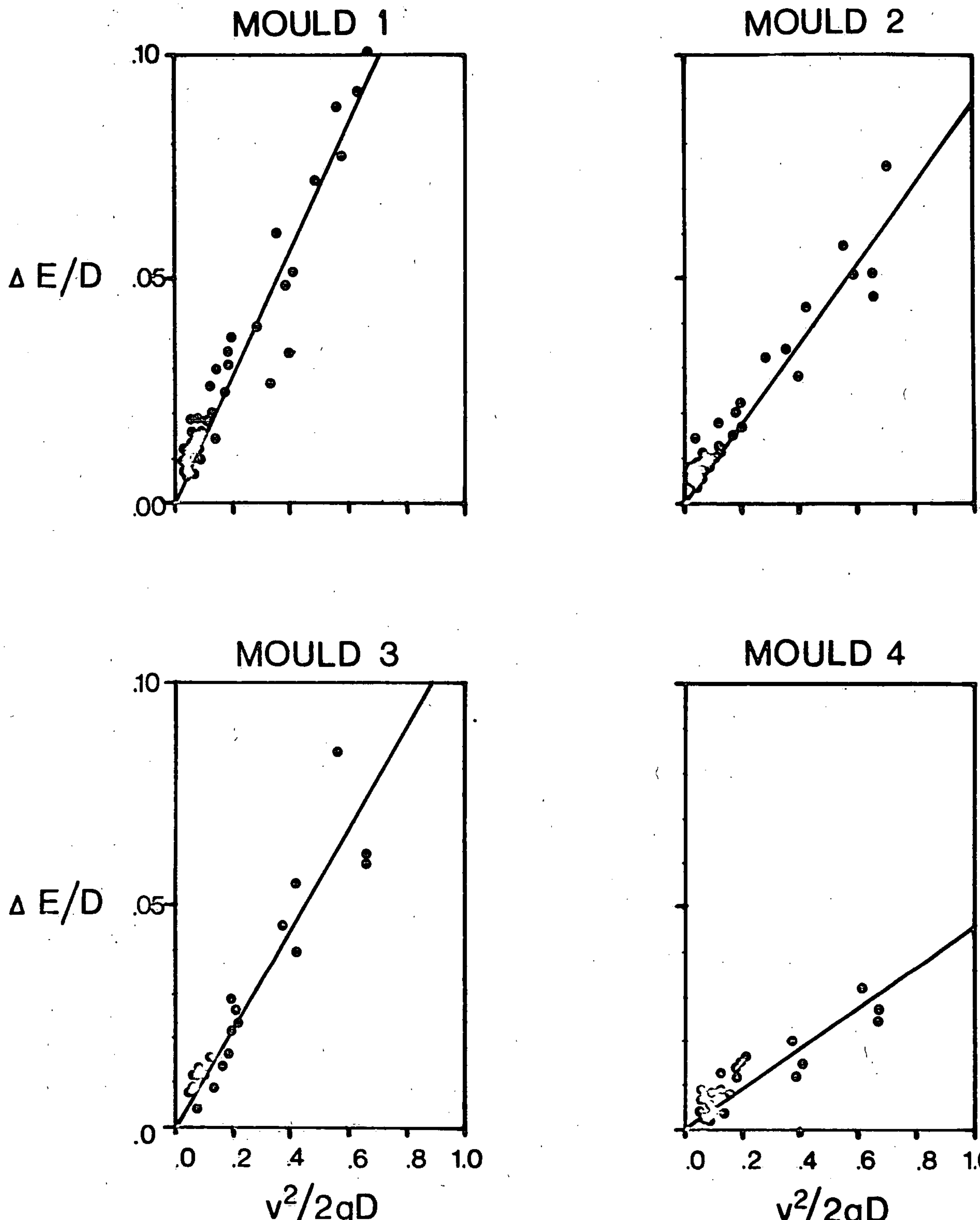


FIG.6 ENERGY LOSSES: CIRCULAR MANHOLE, OPEN-CHANNEL JUNCTION

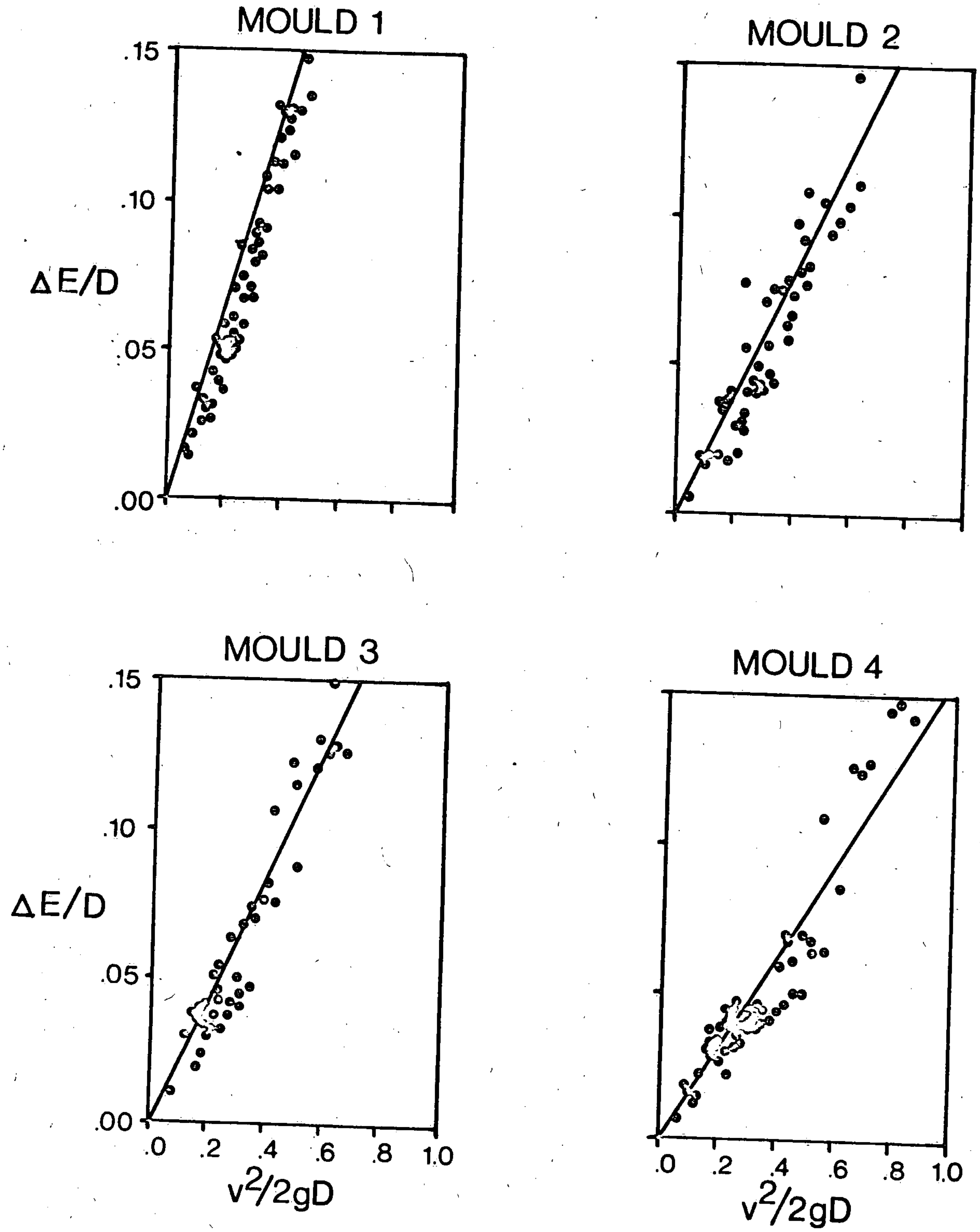


FIG.7 ENERGY LOSS vs. VELOCITY HEAD FOR SQUARE MANHOLE AND PRESSURIZED FLOW

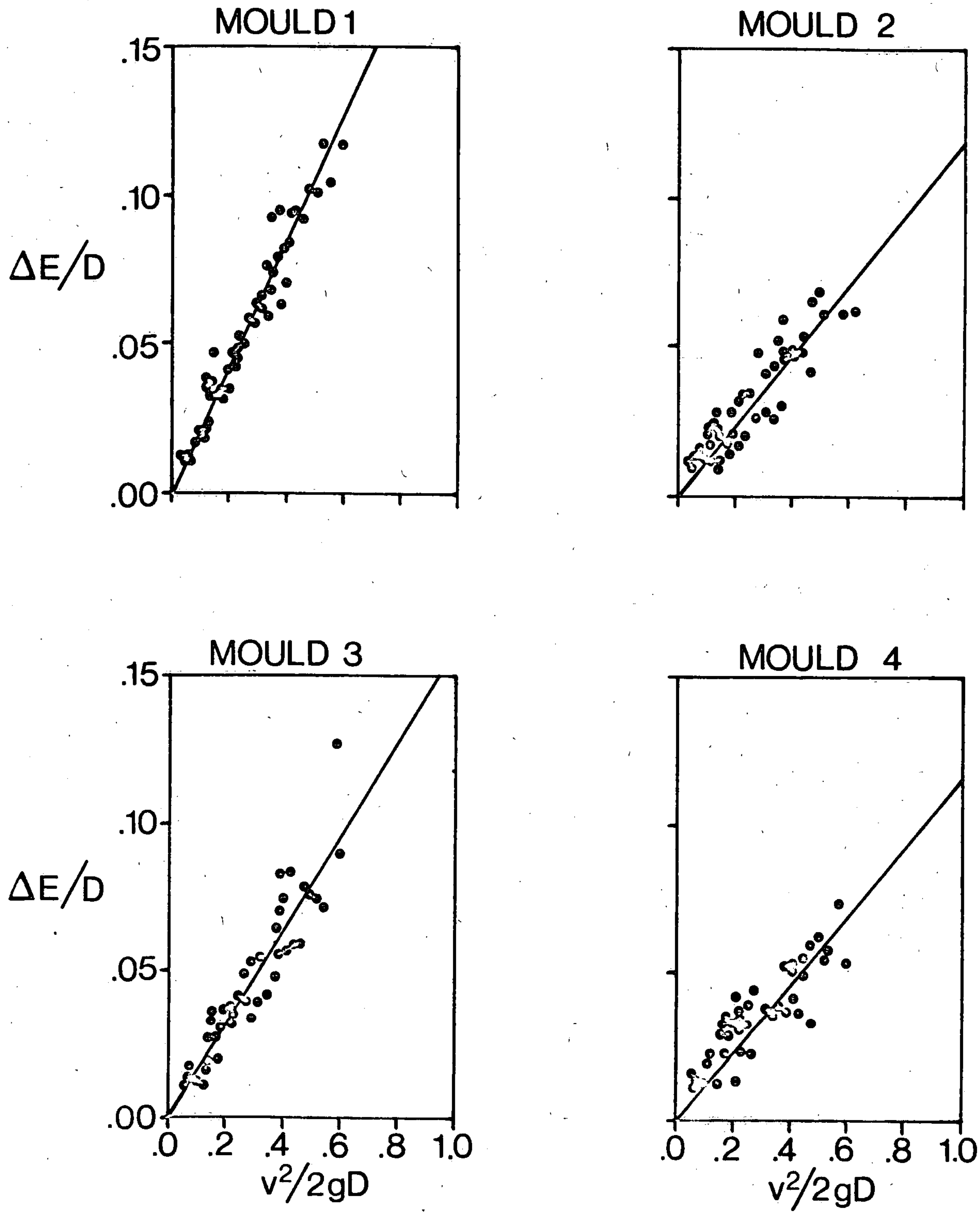


FIG. 8 ENERGY LOSS vs. VELOCITY HEAD FOR CIRCULAR MANHOLE AND PRESSURIZED FLOW

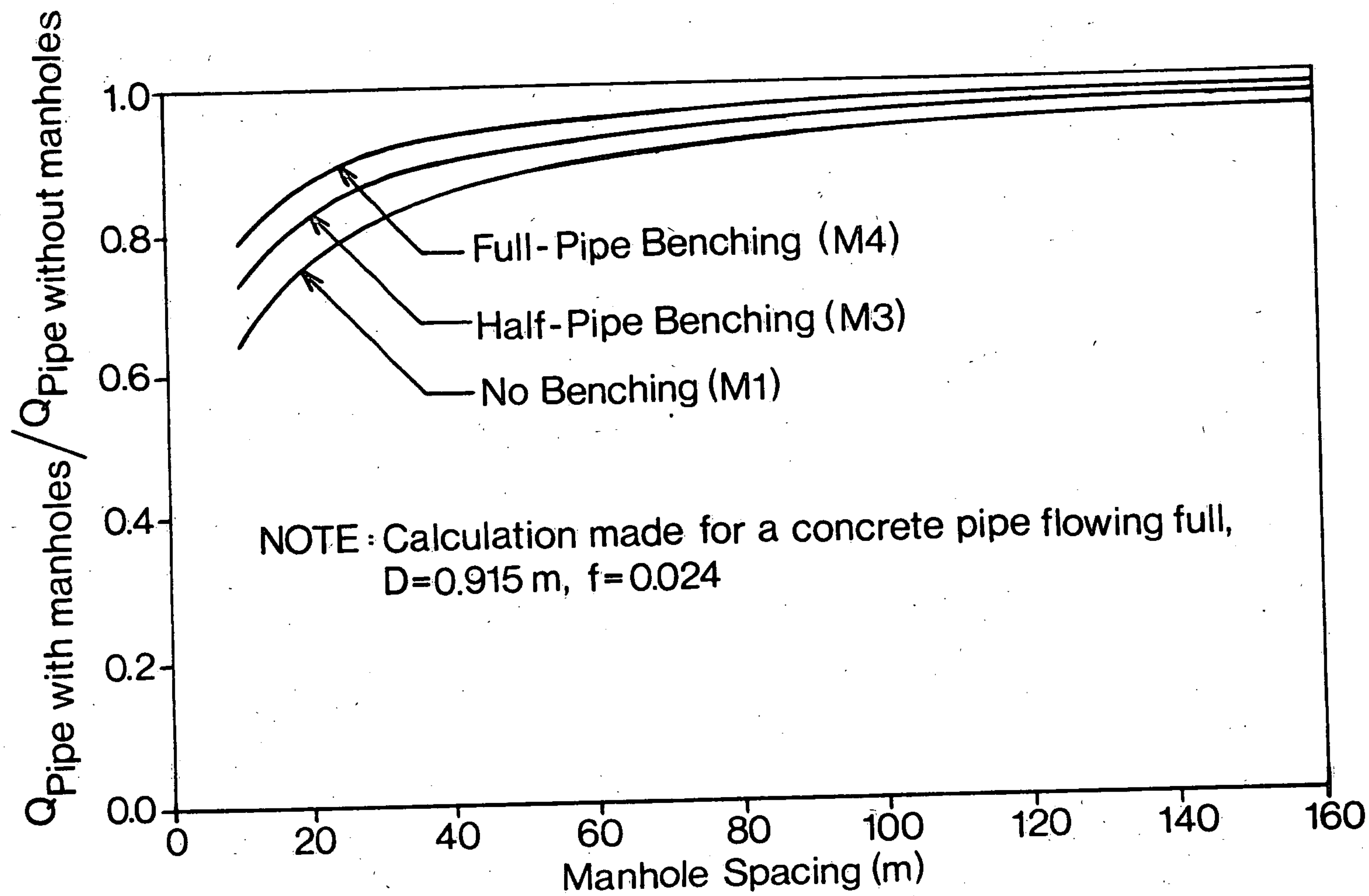


Figure 9 EFFECTS OF MANHOLE SPACING ON SEWER PIPE DISCHARGE