NWRI CONTRIBUTION NO. 85-136

Marsalek (77)

A Contribution to the Civil Engineering Handbook

to be Published by Technomic Publishing, U.S.A.

HEAD LOSSES AT SEWER JUNCTIONS

by

Jiri Marsalek

Environmental Hydraulics Section
Hydraulics Division
National Water Research Institute
Burlington, Ontario L7R 4A6

February, 1986

ABSTRACT

Approaches to design considerations of head losses at sewer junctions are examined. Such approaches include application of the momentum equation and the use of experimental pressure change and head loss coefficients which were reported in the literature. The latter approach, which seems to be both more accurate and practical, is recommended for sewer design. Literature data on pressure changes and head losses at sewer junctions are presented for straight-flow-through junctions, junctions with changes in the pipe alignment, junctions of a main and a lateral, and junctions of two opposed laterals. Other topics briefly discussed include head losses at Y-junctions and four-pipe junctions, considerations of junction head losses in pressurized sewer flow routing, and sulphide gas releases at sewer junctions.

RÉSUMÉ

On examine différentes façons d'aborder les pertes de charge aux raccordements d'égouts. Entre autres, on applique l'équation du momentum et on fait l'essai des coefficients de changements de pression et de pertes de charge qui sont signalés dans la littérature du domaine. Cette dernière façon de procéder, qui semble à la fois plus précise et plus pratique, est recommandée pour le calcul des égouts. Les données que renferme la littérature sur les changements de pression et les pertes de charge aux raccordements d'égouts sont présentées pour des raccordements droits, pour des raccordements avec modification de l'alignement des tuyaux, des raccordements d'une conduite maîtresse et d'un tuyau latéral, et des raccordements de deux tuyaux latéraux opposés. On aborde aussi brièvement d'autres sujets comme les pertes de charge dans des raccordements en Y et à quatre tuyaux, les pertes de charge aux raccordements d'égouts dans des conduites forcées et les émanations de gaz sulfureux aux raccordements d'égouts.

MANAGEMENT PERSPECTIVE

Sewer networks are designed to flood at a designated frequency which has been judged to be acceptable to the community.

Flooding may occur more frequently than calculated if insufficient allowance has been made for pipe losses or alternatively the system may be more expensive than necessary to meet design conditions.

Accurate data on junction losses permits the designer to optimize the collection system for costs and frequency of flooding.

The data on benching shows also what benefits could accrue to an older overloaded system if the junction geometry is improved.

This paper provides data fundamental to sound economic sewer network design.

T. Milne Dick
Chief, Hydraulics Division

PERSPECTIVE-GESTION

Les réseaux d'égouts sont conçus pour déborder à une fréquence déterminée qui a été jugée acceptable pour la collectivité.

Or, les débordements peuvent se produire plus souvent que ce qui avait été calculé si l'on n'a pas suffisamment tenu compte des pertes dans les conduites ou si le système s'avère plus coûteux qu'il n'en faut pour satisfaire aux conditions de la conception.

Des données précises sur les pertes de charge aux raccordements permettent au concepteur d'optimiser les coûts de l'ensemble du système et la fréquence des débordements.

Les données conceptuelles révèlent aussi qu'on tirerait davantage parti d'un vieux système surchargé si on en améliorait la géométrie des raccordements.

Le document fournit des données de base qui se prêtent à un calcul économique et à bon escient des égouts.

Le chef,

T. Milne Dick
Division de l'hydraulique

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

Stormwater, sanitary sewage and combined sewage collection systems consist of sewer pipes and various appurtenances and special structures among which sewer junction manholes are the most common. An example of a layout of a storm sewer system with various types of junction manholes is shown in Fig. 1.

A properly designed sewer system must convey the maximum design flow, transport suspended solids, minimize odour nuisance and meet restrictions on hydraulic grade line elevations in the case of a surcharged system. To meet such design objectives, the sewer network has to be designed as a system in which capacities of individual sewers depend not only on sewer characteristics, but also on flow conditions at manholes and other structures.

The hydraulic design of sewer networks is based on 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, such as manholes. While skin friction losses are caused primarily by viscous and turbulent shears along the conduit boundary, form losses may be caused by shear as well as pressure differentials caused by flow separation, changes in flow alignment, and drag on flow obstructions. Friction head losses have been studied extensively in the past and can be adequately characterized for design purposes. On the other hand, form losses at junction manholes are less well understood and the available information on such head losses is fairly limited. Yet in some cases, form losses at junctions may be fairly large, in comparison to friction losses, and junctions then act as bottlenecks which seriously limit the capacity of the sewer system. Under such circumstances, the sewer system becomes surcharged and this condition may 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 junction head losses need to be considered in sewer design regardless of the design approach taken, the importance of such considerations increased in recent years with the introduction of sophisticated computerized design methods. In the traditional approach, sewer systems are designed as open-channel networks in which the hydraulic grade line does not exceed crown elevations and form head losses are not excessive. Under such circumstances, even crude approximations of junction head losses may be adequate, particularly when dealing with subcritical flows of low velocities (less than 1.5 m/s). 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 occur also in sanitary sewers with high infiltration and inflow. Finally, it is sometimes economical to design deep sanitary sewers (without service connections) for surcharge, or to design storm sewers for surcharge at peak design flow (17). The surcharging of sewer systems is not necessarily harmful as long as the hydraulic grade line does not exceed the critical elevation above which flood damages or overflows occur. A proper design of surcharged systems is based on computerized pressure flow routing through the sewer network and on computations of the hydraulic grade line elevations. The sophistication and accuracy of such calculations is defeated by neglect or improper consideration of junction head losses.

The design of junctions of combined or sanitary sewers should not be limited to hydraulic computations only, but it should also consider sulphide gas releases at junctions. Dissolved sulphide in wastewater tends to pass into air from exposed surfaces. The rate of sulphide gas releases is proportional to the degree of flow turbulence (17) which may be particularly high at sewer junctions. Escaping sulphide, usually in the form of hydrogen sulphide, causes odour nuisances and produces lethal atmosphere in sewers. To minimize such problems, the junction susceptibility to sulphide releases should be also considered in design.

It follows from the preceding discussion that head losses at sewer junctions warrant full consideration in sewer design, particularly when dealing with surcharged systems. Guidance for determination of junction head losses is given in the following sections. The presentation of material starts with a general discussion of the hydraulics of sewer junctions followed by design data for individual types of junctions.

2.0 HYDRAULICS OF SEWER JUNCTIONS

General problems of the flow through channel junctions were discussed by Chow (4) who concluded that the flow through junctions was a rather complicated problem whose generalization by analytical means was not possible and the best solutions would be found by experimental studies of individual junction designs. This suggestion agrees well with the literature survey findings which indicate that experimental investigations, usually done in scale models, are the most common approach to the study of junction hydraulics. Another approach is the application of the momentum equation. Both approaches are discussed later in this section.

As mentioned in the Introduction, sewer junction head losses are particularly important in surcharged sewer systems with pressure flow in sewers but a free water surface at junction manholes. For this reason, the majority of experimental studies dealt only with surcharged sewers and, consequently, the discussion in this chapter also concentrates on problems of junctions of surcharged sewers. Whenever available, additional information on junctions with open-channel flow in all branches is also presented.

The presentation of material in this section starts with basic definitions, followed by application of the momentum equation to sewer junctions, and basic considerations for experimental studies of sewer junctions.

2.1 Basic Definitions

A sketch of flow conditions at a common junction of a main with a perpendicular lateral is shown in Fig. 2. This example is used for a general discussion of flow problems at junctions and introduction of basic definitions.

Flows entering the junction are subject to an energy head loss which comprises various loss components depending on flow conditions. In both pressure and open-channel flows such loss components may include losses at the junction entrance (a sudden expansion), losses due to turbulence inside the junction, losses due to the flow direction change, losses at the outlet (a sudden contraction), and losses due to increased turbulence downstream of the junction. In open-channel flow, additional losses include those caused by flow deceleration upstream of the junction, and flow acceleration and surface waves downstream of the junction. Because it is impractical and often even impossible to separate and evaluate the individual loss components, they are lumped together and referred to as the junction energy head loss, or simply the junction head loss. Such a loss is plotted as a sudden drop in the projected upstream energy grade line above the centre of the junction (see Fig. 2).

In design of surcharged sewer systems, it is not sufficient to make only the energy calculations, but it is also required to calculate the hydraulic grade line elevations for the entire system in order to check whether they meet design restrictions. Consequently, it is necessary to establish pressure changes caused by flow conditions at junctions. For this reason, both head losses and pressure changes are discussed in this chapter.

It is customary to express both the junction head loss, ΔE, and pressure change, ΔP, in terms of the outfall velocity head in the following form:

$$\Delta E = K \frac{v_o^2}{2g} \tag{1}$$

$$\Delta P = K_p \frac{v_o^2}{2g} \tag{2}$$

where K is the head loss coefficient, K_p is the pressure change coefficient, v_o is the outfall mean flow velocity, and g is the acceleration due to gravity. Because both head losses and pressure changes are considered for all inflow pipes, eqs. (1) and (2) are applied to all inflow pipes and appropriate subscripts are introduced for ΔE, ΔP, K and K_p. Most of the discussion in the following sections then concentrates on establishing coefficients K and K_p for various junction designs.

2.2 Applications of the Momentum Equation to Sewer Junctions

Theoretically, the impact head loss at a junction may be computed by application of the momentum principle. This principle states that, for a particular direction x, the sum of external forces (ΣF_x) acting on the junction water body is equal to the change in momentum flux through the junction (4):

$$\Sigma F_x = K_o \rho Q_o v_o - \sum_{i=1}^{n} K_i \rho Q_i v_i \qquad (3)$$

where K is the momentum flux correction coefficient, ρ is the mass density, Q is the branch discharge, v is the mean branch velocity, and subscripts o and i refer to the outfall and inlet branches (i = 1 to n), respectively. External forces include pressure forces at the inlets and outfall, forces transmitted to the fluid from the boundary which confines the flow, and friction forces. Expressions analogous to eq. (3) could be written for other directions.

The pressure head change is introduced into eq. (3) as the difference in pressure heads between the inflow and outfall sections.

In practice, the application of eq. (3) was further simplified by derivation of the so-called Thompson's formula which states that the summation of all pressure forces acting on the junction water body, ignoring friction, is equal to the average cross-sectional area through the junction, multiplied by the change in the hydraulic gradient through the junction (14). The product of the average cross-sectional area and the pressure change then equals the change in the momentum flux.

The accuracy of the momentum equation applications to junction problems may be questioned. In particular, the pressure components, in the direction of flow, along the walls and floor of channels cannot be determined accurately, because of the effects of the impact of streams and the curvature of channels (17). Problems with applications of the momentum equation are further demonstrated below for a surcharged straight-flow-through junction shown in Fig. 3.

Considering the junction shown in Fig. 3, the momentum equation can be written as follows:

$$P_m + P_{uw} - P_{dw} - P_o - F_f = K_o \rho Q_o v_o - K_m \rho Q_m v_m \tag{4}$$

where P_m is the hydrostatic force, F_f is the boundary drag force, K is the momentum correction coefficient, ρ is the mass density, Q is the discharge, v is the mean branch velocity, and subscripts m, o, uw and dw refer to the main, outfall, upstream junction wall and downstream junction wall, respectively. Equation (4) can be further simplified by the following considerations:

(i) $P_{uw} = P_{dw}$, thus $P_{uw} - P_{dw} = 0$.

(ii) The drag forces exerted by junction walls are usually neglected in comparison to pressure forces, thus $F_f \sim 0$.

(iii) It follows from the junction geometry that $Q_m = Q_o = v_o \pi D^2/4$, and $v_m = v_o$.

Using the above simplifications and substitutions, eq. (4) can be written as:

$$\gamma(h-D/2)\frac{\pi D^2}{4} - \gamma\,(h-\Delta P-D/2)\frac{\pi D^2}{4} = (K_o - K_m)\,\rho\,\frac{\pi D^2}{4}\,v_o^2 \qquad (5)$$

where γ is the specific weight, h is the hydrostatic grade line elevation above the junction floor and ΔP is the pressure change at the junction. After solving eq. (5) for ΔP, the following final expression is obtained:

$$\Delta P = 2\,(K_o - K_m)\frac{v_o^2}{2g} = K_p\,\frac{v_o^2}{2g} \qquad (6)$$

where $K_p = 2(K_o - K_m)$.

It is obvious from eq. (6) that if both momentum flux correction coefficients were equal, or neglected as commonly done in engineering applications, eq. (6) would indicate no pressure change at the junction. Yet experimental observations presented later for the junction shown in Fig. 3 clearly indicate a pressure drop at the junction.

The evaluation of the pressure change coefficient K_p by means of eq. (6) would require the knowledge of velocity distributions in the inflow and outfall sections of the junction. From such distributions, K_m and K_o could be determined. In turbulent flow, K typically varies from 1.01 to 1.07 (4). Assuming the lower value for the inflow section and the higher value for the outfall section, the pressure change coefficient would be calculated as $K_p = 2(1.07 - 1.01) \doteq 0.12$. Such a calculated value has a correct order of magnitude in relation to the experimental values.

It appears from the preceding discussion that although the application of the momentum equation offers some insight into the hydraulics of junctions, it does not provide an universal solution because of inherent inaccuracies. Such difficulties with analytical approaches then contributed to the popularity of experimental studies as the most common approach to investigations of junction hydraulics.

2.3 Experimental Studies of Sewer Junctions

Head loss and pressure change coefficients used in sewer design have been typically determined from observations at actual or model sewer junctions. Because of difficulties with observations at actual sewer installations (1), almost all experimental observations have been done in laboratory scale models of sewer junctions. Before presenting the results of such studies, it is desirable to examine experimental variables, their practical range of values, and scaling of model data to the prototype scale.

2.3.1 Experimental variables

In experimental studies of sewer junctions, it is useful to start with a dimensional analysis of the problem. Considering the surcharged sewer junction shown in Fig. 2, the head loss coefficient K can be expressed as a function of the following variables:

$$K = f_1(\rho, \mu, g, Q_o, Q_\ell, S, a, b, D_m, D_\ell, D_o) \qquad (7)$$

where f_1 is a function, ρ is the fluid density, μ is the fluid viscosity, g is the acceleration due to gravity, Q_o is the outlet pipe discharge, Q_ℓ is the lateral pipe discharge (the main pipe discharge $Q_m = Q_o - Q_\ell$), S is the water depth at the junction, a is the junction length, b is the junction width, and D_m, D_ℓ, D_o are the diameters of the main, lateral and outlet pipes, respectively. Dimensional analysis then yields the following expression for the head loss coefficient:

$$K = f_2\left(\frac{Q_o}{g^{1/2}D_o{}^{5/2}}, \frac{\rho Q_o}{\mu D_o}, \frac{Q_\ell}{Q_o}, \frac{S}{D_o}, \frac{a}{D_o}, \frac{b}{D_o}, \frac{D_m}{D_o}, \frac{D_\ell}{D_o}\right) \tag{8}$$

Equation (8) can be further modified by substituting $v_o = 4Q_o/\pi D_o^2$ and $\nu = \mu/\rho$ and by adding the junction benching to the list of independent variables. For simplicity, the benching geometry is described by a dimensionless factor B to avoid the introduction of several geometrical parameters needed to describe various benching shapes. After these modifications, eq. (8) can be written as

$$K = f\left(\frac{v_o}{\sqrt{gD_o}}, \frac{v_o D_o}{\nu}, \frac{Q_\ell}{Q_o}, \frac{S}{D_o}, \frac{a}{D_o}, \frac{b}{D_o}, \frac{D_m}{D_o}, \frac{D_\ell}{D_o}, B\right) \tag{9}$$

Note that eq. (9) applies to junctions with a rectangular base. For junctions with a round base, the terms a/D_o and b/D_o would be replaced by a single term D_{mh}/D_o, where D_{mh} is the manhole diameter.

Similar procedures would be used to derive expressions for a more general case with three inflow pipes for which no experimental data were found in the literature survey. Note also that an expression analogous to eq. (9) would be obtained for the pressure change coefficient K_p.

A discussion of independent variables listed in eq. (9) follows. This discussion focusses on the importance of individual variables in sewer junction design.

The first term, $v_0/\sqrt{gD_0}$ is the Froude number written for the outfall. Earlier studies indicate that for surcharged sewers K does not depend on the Froude number (13,16). On the other hand, the operation of junctions with open-channel flow is affected by the Froude number of branch flows (15). Thus, the Froude number may be neglected in design of surcharged sewer junctions, but it should be considered in design of junctions with open-channel flow in branch pipes.

The second term, $R = v_0D_0/\nu$, is the Reynolds number written for the outfall. Experimental studies indicate that K does not depend on the Reynolds number if R is greater than 10^4 (3,5). This condition is always met in practice, because specifications of the minimum pipe diamter (0.3 m) and the minimum flow velocity (0.61 m/s) lead to Reynolds numbers greater than 10^5. In order to eliminate R from further considerations as an independent variable, it is necessary to undertake scale model studies for R greater than 10^4.

The third term is the relative lateral inflow. This term defines implicitly the relative main inflow as $Q_m/Q_0 = 1 - Q_\ell/Q_0$. In junctions with more than one lateral, more than one lateral inflow would need to be considered. Experimental studies indicate that relative inflows are very important variables affecting junction head losses and pressure changes (15,16).

The fourth term is the relative junction submergence indicating the depth of water at the surcharged junction. This term is important for operation of certain junctions, such as the straight-flow-through junction manholes with a round base (9,10). For all other junction designs discussed later, no evidence of the effects of submergence on K or K_p was found in the literature (16).

The remaining five independent variables in eq. (9) are geometrical terms. The first two, a/D_o and b/D_o describe the junction width and length. For round manholes, these two terms can be replaced by a single term D_{mh}/D_o. The manhole width and length have some influence on junction head losses. Such influences were observed in some laboratory studies in which both these parameters were varied over a very wide range of values (10,15,16). From the practical point of view, the variations in relative manhole sizes are somewhat limited. Considering the most commonly used standard manhole, which is a 1.22 m (4 ft) prefabricated round-base manhole, the pipe sizes applied with this manhole range from 0.2 m to 0.61 m and the corresponding D_m/D_o ratio varies from 2 to 6. Experimental investigations indicate that the effects of the relative manhole size on K are limited for D_{mh}/D_o or b/D_o greater than 2.0 (16).

The next two terms, D_m/D_o and D_ℓ/D_o, indicate the relative sizes of inflow pipes. These terms are important in considerations of head losses because, in conjuction with relative inflows, they determine the momentum fluxes into the junction. Such fluxes then strongly affect head losses and pressure changes at the junction (16). For junctions with more than one lateral, additional relative lateral sizes would be introduced.

Finally, the last term, which is denoted as benching, refers to channels inside the junction. The main function of such channels is to guide flows smoothly through the junction and thereby to reduce head losses. Horizontal benches incorporated in such designs are helpful for maintenance operations. Benchings were found very important in considerations of head losses at sewer junctions (13). Although details of junction benchings are given later, a simple classification of practical benching designs is given here. For better understanding of descriptions of individual designs and their applications to various types of functions, reference is made to Figs. 4, 5, 10 and 16.

Benching B1 represents the simplest design in which no benching or flow guidance is provided at the junction. This design is expected to produce the largest head losses and for that reason should be avoided in practical design.

Benching B2 is formed by extending the lower half of inflow pipes through the junction and adding horizontal or slightly sloping benches at the top of the semicircular channel. This design is used in municipal design practice and should yield lower losses than B1.

Benching B3 is an improved version of design B2. This improvement is achieved by extending the semicircular channels vertically to the pipe crown, thus forming an U-channel, and placing the benches at that level. This design should provide a better flow guidance and lower head losses than B2.

The last design considered, B4, is a special design attempting to further reduce head losses. In principle, it is the B3 design with an expanded flow cross-section through the junction. For this purpose, pipe expanders and reducers are installed upstream and downstream of the junction. Because of increased costs of this design, its use would be limited to critical cases where large reductions of junction head losses are needed (13).

In summary, the eleven independent variables listed in eq. (9) affect junction head losses to various degrees. For surcharged junctions, the most important hydraulic variables seem to be the relative inflows. Among the geometrical parameters, junction benchings and branch pipe diameters are particularly important. The effects of the remaining variables on junction head losses or pressure changes are relatively limited. For open channel flow junctions, the considerations of independent variables would be even more complicated. In eq. (9), the relative submergence would be irrelevant, but additional terms, such as the depths of flow in branches and Froude numbers for branches, would have to be added.

2.3.2 Scaling of Model Results

As mentioned earlier, almost all experimental data on head losses at sewer junctions were observed in scale models. Consequently, it is necessary to establish whether such observations are affected by the scale of the model and whether they need to be scaled up for applications to the prototype.

In general, head loss or pressure change coefficients measured in model junctions are directly transferable to the prototype, if the dimensionless parameters listed in eq. (9) are identical for both the model and the prototype. Although the identity of all parameters listed in eq. (9) is not feasible in all cases, an approximate identity can be attained for special cases discussed below.

For many cases, eq. (9) may be substantially simplified. For example, for surcharged junctions with high Reynolds numbers (greater than 10^4) and higher surcharges ($S/D_o>1.3$), the list of independent variables in eq. (9) may be reduced to relative inflows and junction geometric parameters. In such cases, if the relative inflows (Q_ℓ/Q_o's) in the model and prototype are identical and the model is geometrically similar to the prototype, the head or pressure change coefficients observed in the model are directly transferable to the prototype.

Scaling effects were studied for manholes with a 90° bend using two different size models. On the average, the head losses coefficients observed in both models differed by less than 4% (13). Considering uncertainties involved in such observations, no scaling effects were detected.

Considerations of model similarity for open-channel flow junctions are more complicated than in the former case, because of additional variables affecting the operation of such junctions. The findings reported in the literature (15) indicate that the gravity forces tend to dominate the junction flow processes and that the viscous forces may be neglected. Consequently, the Froude similarity would apply. For this similarity, the head and pressure change coefficients are directly transferable from the model to the prototype for identical Froude numbers in the model and prototype.

3.0 DESIGN DATA

Considering the preceding discussion of the hydraulics of sewer junctions, experimental data from junction models represent the best source of information for evaluation of junction head losses and pressure changes. Such data have been compiled from numerous sources and presented in this section. Wherever required, the literature data are further interpreted and extrapolated to enhance their usefulness for practical design.

Sewer junctions are classified here into the following four categories:

Two-Pipe Junctions

Three-Pipe Junctions

Four-Pipe Junctions

Special Junction Structures.

Each of the above categories has a number of subcategories which are discussed later. The presentation of data starts with the description of data sources, followed by data listings and the listing of recommended design values.

3.1 Two-Pipe Junctions

Two-pipe junctions are the simplest junctions characterized by a single inflow pipe, referred to as the main, and a single outfall pipe. These junctions can be further divided into straight-flow-through junctions without any change in the pipe alignment and junctions with a change in flow alignment. Further discussion of both junction types follows.

3.1.1 Straight-flow-through junctions

Straight-flow-through junctions are shown schematically in Fig. 4. In the simplest form, sewer pipes upstream and downstream of the junction manhole maintain the same alignment and pipe diameter. Such manholes are installed for maintenance purposes, or where the pipe slope changes.

Head losses and pressure changes at straight-flow-through junctions are primarily caused by changes in the flow channel geometry at the junction. For surcharged junctions, the head loss coefficient can be expressed as

$$K = f(S/D_o,\ b/D_o(\text{or } D_{mh}/D_o),\ B) \tag{10}$$

For open-channel flow junctions, the functional expression would be similar to eq. (10), but the head losses are so small that they hardly warrant a detailed consideration.

The literature survey identified six sources of data on straight-flow-through junctions. Some of the reported data were observed in junctions of a main and lateral as a limiting case with no lateral inflow. A brief description of data sources is given in Table 1.

It appears from the above sources that data for straight-flow-through junctions are fairly extensive and consistent. In general, the reported head loss coefficients ($K = K_p$, for $D_m = D_o$) vary from 0.05 to 0.35 for junctions without special flow phenomena present at the junction. The findings regarding various variables affecting head losses and pressure changes at straight-flow-through junctions are summarized below.

In pressurized flows, the head loss or pressure change coefficient varies with the surcharge depth, base shape, manhole width, and benching. The effects of the surcharge depth are perhaps the least understood. For smaller surcharge depths ($S/D_o<2$), these depths seem to affect head losses and pressure changes by inducing the formation of special flow patterns at the junction. The formation of these patterns is further affected by the manhole width, base shape, and benching. In particular, for round-base manholes with a half-pipe benching (B2) and certain ranges of relative manhole widths and depths of surcharge, the formation of strong swirls or periodic sloshing inside the junction was reported (7,9,10). Such phenomena increased the head loss coefficient values to levels as high as 0.9. The highest values were observed for $S/D_o = 1.5$ (9) and they dropped to normal values (<0.35) for higher depths of surcharge ($S/D_o>2$).

Similar phenomena were observed even for higher benchings places asymmetrically inside wide junction manholes (7).

From the practical point of view, sudden increases in head losses at straight-flow-through junctions can be prevented by using a full pipe benching (B3) placed symmetrically inside the junction manhole. In any case, the sudden increases in head losses represent a transient phenomenon which would increase the junction surcharge depth which then leads to the breakdown of the junction swirl or sloshing and concomitant reduction in head losses.

The effects of the manhole base shape are not significant except for the fact that the round base manholes are clearly more susceptible to the formation of junction swirls or sloshing (7).

Head losses seem to increase slightly with the relative manhole width (10,16). Larger manholes are more susceptible to the formation of swirls or sloshing motion.

The benching has a strong influence on head losses at sewer junctions (11). Junctions without any benching exhibit the highest head losses. Such losses can be substantially reduced by installing a full pipe benching (B3) at the junction. This design also seems to avoid the formation of swirls or sloshing motion at the junction.

Head losses in open-channel flow junctions are generally much lower than those at surcharged junctions. Limited data were found in the literature for subcritical flows and average depths of flow. When dealing with supercritical flows, it is required to check whether a hydraulic jump forms at the junction. Standard formulas derived from the momentum eq. (4) can be used for this purpose.

Head loss and pressure change coefficients for design of straight-flow-through junctions are presented in Table 2 below.

Finally, the straight-flow-through junctions with a change in the pipe diamater need to be also considered. Since many design criteria do not allow reductions in the pipe diameter in the downstream direction, only the case of pipe expansion ($D_o > D_m$) is of interest. For this case, the pressure change coefficient can be derived from the momentum equation in the following form (16):

$$K_p = 2[1 - (D_o/D_m)^2] \tag{11}$$

Experimental verification of eq. (11) produced an acceptable agreement between the observed and calculated values for D_m/D_o greater than 0.53 and smaller than 1.0.

For $D_m \neq D_o$, the head loss and pressure change coefficients are not equal. The relationship between both coefficients can be derived as

$$K = K_p - 1 + (D_o/D_m)^4 \tag{12}$$

and after substituting from eq. (11), the final expression was K is obtained in the form

$$K = (D_o/D_m)^4 - 2(D_o/D_m)^2 + 1 \qquad (13)$$

Note that for $D_o > D_m$, eq. (11) yields negative K_p's and the pressure grade line elevation increases downstream of the junction. For $D_o \simeq D_m$, eq. (13) yields $K \simeq 0$ and obviously underestimates head losses. This is caused by simplifying assumptions used in the derivation of eq. (11).

3.1.2 Two-pipe junction with a change in alignment

Although the pipe alignment can be changed gradually by using curved sewers, it is much more practical to change the alignment abruptly at a junction manhole. Such changes are described by the deflection angle θ which typically varies from 22.5° to 90°. An example of a two-pipe junction with a 90° bend is shown in Fig. 5.

Head losses at surcharged junctions with a bend are caused not only by changes in the flow cross-section at the junction, but also by changes in the flow direction. For the limiting case of θ = 90°, it appears that the head loss coefficient depends primarily on the junction benching (13). Other factors which were listed in eq. (10) for straight-flow-through junctions, such as S/D_o, b/D_o (or D_{mh}/D_o), and the base shape, have only minor influence.

Another factor affecting head losses at manholes with a bend is the location of the branch point at which the inflow and outfall pipe axes intersect. The lowest losses were found for branch points on the outfall wall (6) as shown in Fig. 6 for two manhole layouts. Both junctions in Fig. 6 have the same deflection angle of 45°, but significantly different pressure change coefficients. The design with the branch point located on the inflow wall has $K_p = 2.20$ ($D_m \doteq D_o$) and the design with the branch point on the outfall wall has $K_p = 0.60$ ($D_m = D_o$). The hydraulic effectiveness of the latter design is obvious.

In summary, it appears that head loss and pressure change coefficients for surcharged two-pipe junctions with a bend depend primarily on the deflection angle, junction benching, and location of the branch point. Junctions with open-channel flow would behave similarly and their head loss would be also affected by the Froude numbers of the inflow and outfall.

A literature survey identified three sources of information on head or pressure changes at junctions with a bend (1,6,13). The basic data from these sources are summarized in Table 3.

It is obvious from Table 3 that none of the existing data sets covers a full range of both major variables, the deflection angle and benching. Consequently, it is necessary to use the existing data for interpolations and extrapolations of missing values. Toward this end, the appropriate data from the references listed in Table 3 were plotted in Fig. 7 and used in interpolations and extrapolations. Such procedures increase uncertainties in the final design data which are presented in Table 4 for various deflection angles and benchings.

It appears from the data in Table 4 that head losses and pressure changes depend strongly on both the deflection angle and benching. The finding about the benching effects is particularly important because it can be used to develop a special low head loss design (13). Such a design, characterized by the full depth benching and expanded flow cross-sections at the junction, is shown in Fig. 8 for $\theta = 90°$. This design produced a low head loss coefficient of 0.65.

The above data were produced for identical pipe diameters upstream and downstream of the junction. The junctions with changes in both pipe alignment and diameter were also studied by some researchers (6,16) and the pertinent data are listed in Table 5.

The data in Table 5 indicate that for high deflection angles and branch points upstream of the outfall wall, there is little variation observed in K_p's. In a conservative approach, it is possible to adopt a constant K_p (for $D_m = D_m$) and apply it to all values of D_m/D_o. Another possiblity is to reduce K_p for lower values of D_m/D_o as indicated by trends in Table 5. The corresponding K would be calculated from eq. (13).

The available experimental data for head losses at junctions with bends and open-channel flow conditions are very limited. The only case studied was a 90° bend and three junction benchings (13).

For subcritical flows and full benching, there are no changes in flow areas at the junction and the head losses arise only from changes in the flow direction.

In comparison to surcharged junctions, losses at junctions with open-channel flow are significantly smaller. Consequently, even crude approximations of the head loss coefficient obtained from interpolations of data for $\theta = 0°$ and $\theta = 90°$ may be acceptable. Both observed and interpolated K's are given in Table 6 and recommended for use in junction design. The coefficients given in Table 6 represent averages for various depths of flow.

3.2 Three-Pipe Junctions

Among three-pipe junctions, the most common are T-junctions of a main and lateral, or junctions of two opposed laterals. Less common are Y-junctions.

A general analysis of surcharged three-pipe junctions indicates that the head loss or pressure change coefficients can be described in the following form:

$$K, K_p = f(Q_\ell/Q_o, \theta, b/D_o \text{ or } D_{mh}/D_o, D_m/D_o, D_\ell/D_o, \text{base shape}, B) \tag{14}$$

where θ is the lateral branch angle.

Thus for pressurized flow the junction head loss or pressure change coefficients depend only on one hydraulic variable, the relative lateral inflow Q_ℓ/Q_o, and several geometric characteristics of the junction and sewer pipes. Note the absence of the junction surcharge depth which had some effects on operation of certain two-pipe junctions. Among the geometric parameters, the most important ones are the benching (13), relative pipe sizes (16), and possibly the lateral branch angle θ. Other geometrical parameters, such as the relative manhole size and base shape, are barely significant.

The discussion of T-junctions starts with junctions of a main with perpendicular lateral followed by junctions with two opposed laterals.

3.2.1 Junctions of a main with lateral

Four sources of data for junctions of a main with perpendicular lateral are listed in Table 7. It appears that such data are fairly extensive and provide a good basis for junction design.

It can be inferred from Table 7 that different studies focussed on different aspects of the junction problem and none of them can serve as a sole source of design data. Consequently, two design aids were prepared - a graph for junctions with various pipe diameters and tables of K_p's for junctions with comparable pipe sizes.

The earlier published design graphs (16) present K_p as a function of the relative lateral discharge Q_ℓ/Q_o and the relative main pipe diameter D_m/D_o. These graphs were prepared for surcharged junctions of a main with perpendicular lateral, and no benching at the junction. The basic graph from Ref. (16) is replotted in Fig. 8. This graph was prepared for junctions with comparable D_m and D_ℓ, and in practical applications, identical pressure change coefficients are assumed for both the main and lateral ($K_{pm} = K_{p\ell}$). It appears from Fig. 9 that K_p increases with D_m/D_o and Q_ℓ/Q_o.

For $D_m/D_o = 1$, the data in Fig. 9 were verified against some recently published data (13). Graphs for junction benchings B2 and B3 (see Figs. 10-12) were produced by extrapolation. Such extrapolated graphs are only approximate, because they are based on observations for $D_m/D_o = 1$ and the general shape of curves shown in Fig. 9. The above extrapolations were done for a limited range of D_m/D_o from 0.7 to 1.0.

The other case of interest is characterized by comparable sizes of the main and outfall, and a similar size or smaller lateral. Such a case was studied for two D_ℓ/D_o's and three benchings (13). Observed K_{pm}'s and $K_{p\ell}$'s were plotted in Figs. 11 and 12 for $D_\ell/D_o = 0.5$ and 1.0, and extrapolated for other values.

The data in Figs. 11 and 12 indicate differences in K_{pm} and $K_{p\ell}$ for smaller lateral pipe diameters. For $D_\ell/D_o = 1$, both K_{pm} and $K_{p\ell}$ are practically identical. For equal lateral discharges, smaller laterals are characterized by a higher flow momentum which leads to a greater flow disruption and pressure changes in the main pipe. For small laterals and low Q_ℓ/Q_o's, the pressure change coefficient for the lateral pipe is negative.

Among other parameters listed in eq. (14), the manhole base shape did not affect K_p's at all. The relative manhole size had some effect on K_p's - the coefficient values slightly increased with the manhole size. Head losses seem to increase with the lateral angle θ. This parameter, however, cannot be fully evaluated because of the lack of supporting experimental data.

Open-channel flow junctions of a main and lateral were investigated in two studies (13,15). Both studies indicated the importance of benching for reduction of head losses. Full pipe depth benchings (B3) resulted in the lowest head losses. It was further found that the hydraulically effective junctions should be as short as practical and the drop between the inflow and outfall inverts should be gradual (15). In junctions without benching, some reduction in head losses was achieved by installing hinged deflector plates (15).

Generally, head losses in open-channel flow junctions are fairly small with most K's being smaller than 0.5. Some guidance for selection of K values for junctions with various benchings can be obtained from Fig. 13. Head loss coefficients in Fig. 13 represent average values obtained for various subcritical flow depths. It should be further noted in Fig. 13 that for low Q_ℓ/Q_o's, the K_ℓ values are negative and the lateral flow experiences an apparent energy gain. Under such circumstances, the water is drawn from the lateral in a manner resembling ejector action. Note, however, that the energy gain is indicated by calculations in which the downstream channel energy is based on the mean flow velocity in this channel. Such a gain is only apparent because the water from lateral enters into the main flow region where the velocity is below the mean value. In any case, negative head losses, or apparent energy gains, are neglected in practical design.

3.2.2 Junctions of two opposed laterals

Junctions of two opposed laterals are characterized by large head losses and pressure changes and, whenever possible, their use in sewer network layouts should be avoided.

Experimental studies indicate that the head loss and pressure change coefficients for surcharged junctions of two opposed laterals are primarily dependent on the relative lateral inflow $Q_{\ell 1}/Q_o$ ($Q_{\ell 2}/D_o$ = 1- $Q_{\ell 1}/Q_o$), the relative lateral sizes $D_{\ell 1}/D_o$ and $D_{\ell 2}/D_o$, benching, and lateral alignment (13,16). In the preceding notation, $\ell 1$ and $\ell 2$ denote the first and second lateral, as shown in Fig. 14.

The pressure change coefficients for two opposed laterals joined at a junction without benching, depend on the relation of flow velocities in both laterals (16). K_p for the higher flow velocity lateral, K_{phv}, is approximately constant throughout a wide range of flows and attains a value of 1.8. For the lower velocity lateral, the pressure change coefficient $K_{p\ell v}$ can be determined as the difference of two pressure factors, H and L, plotted in Fig. 14 (16). Thus, both lateral pressure change coefficients can be determined from the following expressions:

$$\text{Higher velocity lateral} \quad K_{phv} = 1.8 \qquad (15)$$

$$\text{Lower velocity lateral} \quad K_{p\ell v} = H - L - 0.2 \qquad (16)$$

Investigations of junctions of two opposed laterals with benchings indicate that for identical pipe diameters ($D_{\ell 1} = D_{\ell 2} = D_o$) and intermediate flow divisions ($Q_{\ell 1}/Q_o = 0.3 - 0.7$), the benching barely contributes to the reduction of pressure change coefficients (13). For benchings B2 and B3 shown in Fig. 14, K_p's were only 8% and 17%, respectively, smaller than those corresponding to the junction without benching. Thus it appears that the earlier presented data for junctions without benching may be transposed to junctions with benching without introduction of significant inaccuracies.

Experimental studies further indicate that the lowest head losses and pressure changes at surcharged junctions of two opposed laterals are found for junctions with comparable flow velocities in both laterals and full pipe depth benching (13). Another means of reducing pressure changes is to offset laterals at the junction (16). For design of junctions with opposed offset laterals, the design chart in Fig. 15 can be used.

Limited experimental data are available for junctions of two opposed laterals with open-channel flow. Such data were produced for subcritical flow with Froude numbers ranging from 0.1 to 0.6, identical pipe sizes $D_{\ell 1} = D_{\ell 2} = D_0$, the depths of flow from $0.4\ D_0$ to $0.9\ D_0$, and three junction benchings B1-B3. For such conditions, approximate head loss coefficients were calculated as averages for all depths and plotted in Fig. 16. The uncertainties associated with data curves in Fig. 16 are estimated as ±0.1.

It can be inferred from Fig. 16 that the relative inflow and the benching are again fairly important parameters. The highest losses were observed at junctions without benching. Such losses were substantially reduced for benching B2 and even more for B3.

3.2.3 Y-junctions

Although T-junctions are the most common among three-pipe junctions, other layouts, such as Y-junctions are also used in some sewer networks. In these junctions, flows from two laterals combine at the junction and leave through the outfall. In comparison to junctions of opposed laterals, Y-junctions should be hydraulically more effective, because some momentum from both laterals should be preserved at the junction and contributed to the outflow. This would be particularly true when both lateral inflows are comparable in terms of flow rates and velocities.

No data on pressure changes at Y-junctions were found in the literature. It was reported, however, that the junction design can be improved by installing a vertical divider separating both inflows as shown in Fig. 17 (15). This divider should split the flow area in the same ratio as that of the two lateral peak inflows.

3.3 Four-Pipe Junctions

Four-pipe junctions shown schematically in Fig. 18 are rarely used in design practice because of high losses taking place at such junctions. The literature survey did not reveal any published information on these junctions. In the absence of specific design data, the designer has to approach four-pipe junctions using the information available for other junction types. Toward this end, it is suggested first to calculate momentum fluxes (ρQv) for all inflow pipes. After comparing magnitudes of such fluxes, it will be possible in many cases to consider the four-pipe junction as one of three-pipe junctions discussed earlier.

For a relatively low momentum flux in the main pipe, the four-pipe junction may be considered as a junction of two opposed laterals discussed in the preceding section. The higher of the two lateral pressure grade line elevations would be assumed for the main pipe as well. When either of the two laterals has a low momentum flux, the four-pipe junction may be considered as a T-junction of a main with perpendicular lateral. Only when all three inflow momentum fluxes are comparable, the above simplifications are not realistic, although they may represent the only solution available to the designer.

Inferences from the earlier discussed studies of T-junctions indicate that it would be desirable to avoid high head losses and pressure changes expected at four-pipe junctions with in-line laterals. This could be done by offsetting the laterals, either within the junction manholes, or by inserting another junction manhole and replacing the four-pipe junction by two T-junctions of a main and lateral.

3.4 Special Junction Structures

The previous discussion dealt primarily with common sewer junctions and junction manholes which are often prefabricated and can be characterized by relatively small dimensions and flows. In large metropolitan sewer networks, there may be cases where several large trunk sewers are joined at special junction chambers. Other special structures comprise various drop manholes which again can have large dimensions. Considering the high costs and importance of such special structures, it is desirable to investigate their operation and design in detail. Such investigations include a computational analysis and possibly a scale model study designed to develop hydraulically effective layouts of special structures. Because the flow conditions at such structures are dominated by gravity effects, the scale models would be typically designed and operated according to the Froude similarity.

4.0 APPLICATIONS OF JUNCTION HEAD LOSSES IN PRESSURIZED SEWER FLOW ROUTING

As stated in the introduction, junction head losses and pressure changes are particularly large and important in surcharged sewer systems. The analysis of surcharged sewer networks differs from that of fully pressurized water distribution networks, because there is a free water surface at junctions and possible overflows (flow losses) at various structures.

Flow routing in surcharged sewer networks requires calculation of flow velocities in individual pipes and calculations of pressure grade line elevations throughout the network. For such calculations, it is important to consider head losses and pressure changes at junctions. Since flow routing calculations are usually computerized, head loss or pressure change coefficients can be used as inputs to the flow routing model. These coefficients may depend on certain flow characteristics and their variations can be described using the design data from the preceding section.

If the routing model does not consider explicitly junction head losses or pressure changes, such factors can be still considered implicitly by using the equivalent roughness concept (12). In this concept, the pipe roughness is increased in calculations to compensate for the form losses at the adjacent junction. This condition can be expressed as

$$H_{eq} = H_j + H_p \tag{17}$$

where H_{eq} is the equivalent pipe head loss, H_j is the junction head loss, and H_p is the actual pipe head loss.

After substituting for $H_j = Kv^2/2g$ and $H_p = Lv^2n^2(D/4)^{-4/3}$, from the Manning equation, the following expression is obtained for the equivalent pipe roughness:

$$n_{eq} = \left(n^2 + \frac{K}{2gL}\left(\frac{D}{4}\right)^{4/3}\right)^{1/2} \tag{18}$$

where n_{eq} is the equivalent (increased) Manning's roughness coefficient, K is the junction head loss coefficient, D is the pipe diameter, L is the pipe length, and n is the actual pipe roughness coefficient. Note that the junction head loss could be also compensated for by lengthening the pipe, but this may lead to problems with numerical solutions employed in some routing models (12).

Approximations of junction head losses by equivalent pipe losses are quite appropriate in cases where the junction coefficient K does not vary strongly with such flow characteristics as the depth of surcharge, or the relative lateral inflows. These characteristics will vary during the passage of surcharge waves through the sewer network and the corresponding changes in K cannot be reflected by the equivalent pipe roughness which remains constant during computations. Under such circumstances, it may be necessary to use iterations in order to establish the flow characteristics corresponding to the peak flow and then use the appropriate values of K and equivalent pipe roughness in final computations.

5.0 SULPHIDE GAS RELEASES AT JUNCTIONS

A proper design of sewer systems receiving discharges of sanitary sewage must consider sulphide gas releases which are closely related to flow conditions in the system. Although sulphide gases may escape from any exposed surfaces of the sewage flow, such releases are strongly enhanced by flow turbulence which is particularly strong at sewer junctions. In order to minimize sulphide gas releases, which create odour nuisances and safety problems for personnel entering sewers, it is desirable to use hydraulically effective junction designs with low levels of turbulence.

Ratings of susceptibility of selected junction designs to sulphide gas releases are presented in Table 8 (13).

It is obvious from Table 8 that where sulphide releases are of concern, the use of hydraulically effective junction designs with full pipe depth benching is required. It was further noted that lower turbulence and sulphide releases can be expected for higher depths of surcharge and comparable magnitudes of inflows (13).

6.0 SUMMARY

Head losses and pressure changes at sewer junctions are significantly large to affect the system discharge capacity. Junction flow phenomena are particularly important in surcharged sewer systems with higher flow velocities as well as higher head losses, and design requirements for calculation of pressure grade line elevations. Therefore, it is a good design practice to evaluate head losses at sewer junctions and to compensate for these losses by equivalent invert drops at the junction. A gradual lowering of the invert elevation is more effective than sudden drops.

Head losses and pressure changes at junctions are affected by a number of hydraulic and geometric variables. The designer's control over flow variables, such as discharges in branch lines, is usually limited and, consequently, it is important to concentrate on the geometrical design of junctions. The most important parameter in this regard is the junction benching. Hydraulically effective junctions should be designed with benching extending to the pipe crown (B3). In the plan, such channels should provide for a gradual change in the flow direction, merging of incoming flows, and deflection into the outfall. In critical cases, a special design incorporating an enlarged pipe immediately upstream of the junction and a correspondingly wide and deep U-channel benching at the junction can be used. Such installations will be more costly.

In existing sewer systems with surcharge problems, the retrofitting of junctions with proper benching should be considered as one of remedial measures. For this purpose, fibreglass inserts or concrete benchings can be used.

Other geometrical parameters of junction manholes are less important. In terms of head losses, no significant differences were found between square- or round-base manholes. Round-base manholes may be more susceptible, in special cases, to formation of swirls or lateral water surface oscillations at the junction which then contribute to increased head losses.

The relative size of manholes has a small influence on head losses. Larger manholes, compared to the outfall diameter, produce somewhat higher head losses. For maintenance purposes, it is desirable to maintain a certain minimum manhole size, in relation to adjacent sewers, to allow some working space. In practice, the minimum value of D_{mh}/D_0 is about two.

In the vertical arrangement, it is common to match pipe crown levels at the junction. The junction invert should be sloping in the flow direction to compensate for the maximum head loss. For pipes with smaller losses, or for lower flows, there may be an energy surplus at the junction.

The list of flow variables affecting junction operations includes the relative surcharge, relative inflows, and the Froude number for open-channel flow. For open-channel flow, junctions behave somewhat differently than in pressurized flow. If the flow is supercritical, hydraulically ineffective junctions may cause hydraulic jumps with concomitant increases in the flow depth and possible sewer surcharging. Such disruptions should be avoided. In subcritical flow, head losses at junctions are fairly small because of relative small changes in the flow area at the junction.

The relative surcharge depth may affect head losses at some sewer junctions. Among the junction types discussed in Section 3, only the straight-flow-through junctions were affected by the surcharge depth for S/D_o smaller than two. Such junctions, with a round base and without benching, were susceptible to formation of swirls and water oscillations at the junction and this resulted in a sudden increase of head losses.

The relative junction inflows are usually given by the overall project layout and their control may not be within designer's power. It is possible, however, to affect the momentum flux of incoming flows by varying the pipe diameter. In this regard, it is important to reduce the velocity of the stream disturbing the main flow through the junction. For example, in a T-junction of a main with lateral, it is desirable to reduce the lateral velocity by using a larger lateral pipe. Similarly, it may be sometimes advantageous to increase the pipe diameter upstream of the junction. Although the head loss coefficient is generally not affected by the pipe diameter, the loss magnitude expressed as $Kv /2g$ will change substantially with the pipe diameter and the resulting change in flow velocity. For example, by enlarging the pipe diameter 1.2 times, the velocity head and the head loss are reduced in half.

Hydraulically effective junction designs with full pipe depth benching are also effective in reducing sulphide releases caused by high turbulence at junctions. Such considerations are important in sewer systems receiving sanitary sewage discharges.

7.0 REFERENCES

1. Ackers, P. An Investigation of Head Losses at Sewer Manholes. Civil Engineering and Public Works Review, Vol. 54, No. 637, 1959, pp. 882-884 and 1033-1036.

2. Archer, B., Bettes, F. and Colyer, P.J. Head Losses and Air Entrainment at Surcharged Manholes. Report No. IT185, Hydraulics Research Station, Wallingford, 1978.

3. Black, R.G. and Pigott, T.L. Head Losses at Two Pipe Stormwater Junction Chambers. Proc. Second National Conf. on Local Government Engineering, Brisbane, September 19-22, 1983, pp. 219-223.

4. Chow, V.T. Open Channel Hydraulics. McGraw-Hill, New York, 1959.

5. de Groot, C.F. and Boyd, M.J. Experimental Determination of Head Losses in Stormwater Systems. Proc. Second National Conf. on Local Government Engineering, Brisbane, September 19-22, 1983.

6. Hare, C.M. Magnitude of Hydraulic Losses at Junctions in Piped Drainage Systems. Civil Engineering Transactions, Institution of Civil Engineers, 1983, pp. 71-77.

7. Howarth, D.A. and Saul, A.J. Energy Loss Coefficients at Manholes. Proc. 3rd Int. Conf. on Urban Storm Drainage, Goteborg, June 4-8, 1984, pp. 127-136.

8. Jensen, M. Hydraulic Energy Relations for Combined Flow and Their Application to the Ninety Degree Junction. Series Paper No. 11, The Royal Veterinary and Agricultural University, Copenhagen, 1981.

9. Liebman, H. Der Einfluss von Einsteigschachten auf den Abflussforgang in Abwasserkanalen (Effects of Manholes on Flow Processes in Storm Drains), Wasser und Abwasser in Forschung und Praxis, Band 2, Erich Schmid Verlag, Bielefeld, FRG, 1970.

10. Lindvall, G. Head Losses at Surcharged Manholes With a Main Pipe and a 90° Lateral. Proc. 3rd Int. Conf. on Urban Storm Drainage, Goteborg, June 4-8, 1984, pp. 137-146.

11. Marsalek, J. Head Losses at Sewer Junction Manholes. Journal of Hydraulic Engineering, ASCE, Vol. 110, 1984, pp. 1150-1154.

12. Marsalek, J. Urban Runoff Peak Frequency Curves. Nordic Hydrology, Vol. 15, 1984, pp. 85-101.

13. Marsalek, J. Head Losses at Selected Sewer Manholes. A report submitted to APWA, National Water Research Institute, Burlington, Ontario, Report No. 85-15, July, 1985.

14. Pardee, L.A. Hydraulic Analysis of Junctions. Office Standard No. 115, Storm Drain Design Division, City of Los Angeles, 1968.

15. Prins, R.J. Storm Sewer Junction Geometry and Related Energy Losses. M.A.Sc. Thesis, University of Ottawa, Ottawa, December, 1975.

16. Sangster, W.M., Wood, H.W., Snerdon, E.T. and Bossy, H.G. Pressure Changes at Storm Drain Junctions. Engineering Series Bull. 41, Engng. Exp. Station, University of Missouri, Columbia, Missouri, 1958.

17. Water Pollution Control Federation. Design and Construction of Sanitary and Storm Sewers. WPCF Manual of Practice No. 9, Water Pollution Control Federation, Washington, D.C., 1970.

TABLES

Table 1. Sources of Data on Head Losses at Straight-Flow-Through Junctions

Author	Type of Flow	Benching	Reference
Archer et al.	P[1]	B2	2
Howarth and Saul	P	B2, special design	7
Liebmann	P, OC[2]	special design	8
Lindvall	P	B2	10
Marsalek	P,OC	B1, B2 and B3	11
Sangster et al.	P	B1	16

[1] Pressurized flow
[2] Open-channel flow

Table 2. Head Loss and Pressure Change Coefficients for Straight-Flow-Through Junctions ($D_o = D_m$)

Junction Manhole				
Base Shape	Relative Size b/D_o, D_{mh}/D_o	Benching	Flow	K (= K_p for Pressurized Flow)
Square, Round	2-5	B1	Pressurized	0.15-0.30[1]
Square, Round	2-5	B2	Pressurized	0.15-0.25[1,2]
Square, Round	2-5	B3	Pressurized	0.10-0.15[1],
Square, Round	2-5	B1	Open-Channel	0.15[3]
Square, Round	2-5	B2	Open-Channel	0.10[3]
Square, Round	2-5	B3	Open-Channel	0.05[3]

[1] Lower values should be used for small b/D_o's or D_{mh}/D_o's and vice versa.

[2] For round-base manholes, this value may increase up to 0.9 for the range of submergence depths from 1.2 to 2.0.

[3] Approximate values obtained as averages of observations for various depths and a subcritical flow.

Table 3. **Sources of Data on Head Losses and Pressure Changes at Junctions With a Bend ($D_m = D_o$)**

Author	Type of Flow	Junction Characteristics: θ	Base	b/D_o (D_{mh}/D_o)	Benching	$K(= K_p)$	Reference
Archer	P[1]	30°	RC[3]	5	B3	0.40	2
et al.	P	60°	RC	5	B3	0.85	2
	P	30°	Rd[4]	5	B3	0.50	2
	P	60°	Rd	5	B3	0.95	2
Hare	P	22.5°	Sq[5]	2	B1	0.30	6
	P	45°	Sq	2	B1	0.60	6
	P	45°	Sq	2	B1	2.20	6
	P	67.5°	Sq	2	B1	2.00	6
	P	90°	Sq	2	B1	1.85	6
Marsalek	P	90°	Sq,Rd	2.3,4.6	B1	1.75	13
	P	90°	Sq,Rd	2.3,4.6	B2	1.65	13
	P	90°	Sq,Rd	2.3,4.6	B3	1.10	13
	P	90°	Sq,Rd	2.3,4.6	B4	0.65	13
	OC[2]	90°	Sq,Rd	2.3,4.6	B1	1.10	13
	OC	90°	Sq,Rd	2.3,4.6	B2	0.60	13
	OC	90°	Sq,Rd	2.3,4.6	B3	0.30	13

[1] Pressurized flow.
[2] Open-channel flow.
[3] Rectangular.
[4] Round.
[5] Square.

Table 4. Head Loss and Pressure Change Coefficients for Surcharged Junctions with a Bend ($D_m = D_o$)

	Deflection Angle θ								
	30°			60°			90°		
Benching	B1	B2	B3	B1	B2	B3	B1	B2	B3
$K = K_p$	0.90	0.80	0.50	1.35	1.25	0.85	1.85	1.65	1.10

Table 5. **Pressure Change Coefficients for Junctions With Changes in Pipe Alignment and Diameter (After Refs. 6 and 16)**

	K_p						
	Deflection Angle						
D_m/D_o	22.5°	45°[1]	45°[2]	67.5°	90°	90°[3]	90°[3]
0.52						1.71	1.91
0.65						2.20	1.87
0.70	-1.6	-0.9	2.05	1.70	1.50		
0.80	-0.6	0.0	2.10	1.80	1.65		
0.83						2.31	2.20
0.90	0.0	0.45	2.15	1.90	1.75		
1.00	0.30	0.60	2.20	2.00	1.85	2.00	2.00

1 Branch point on the outfall wall.
2 Branch point on the inflow wall.
3 Data from Ref. 16 for very narrow manholes. All other data were adopted from Ref. 6.

Table 6. Head Loss Coefficients for Junctions Wtih Bends and Subcritical Open-Channel Flow ($D_m = D_o$)

	K											
	Deflection Angle θ											
	30°			60°			90°					
Benching	B1	B2	B3	B1	B2	B3	B1	B2	B3	B1	B2	B3
K	0.15	0.10	0.05	0.47	0.27	0.13	0.79	0.44	0.21	1.10	0.60	0.30

Table 7. Sources of Data on Head Losses and Pressure Changes at Junctions of a Main With Perpendicular Lateral

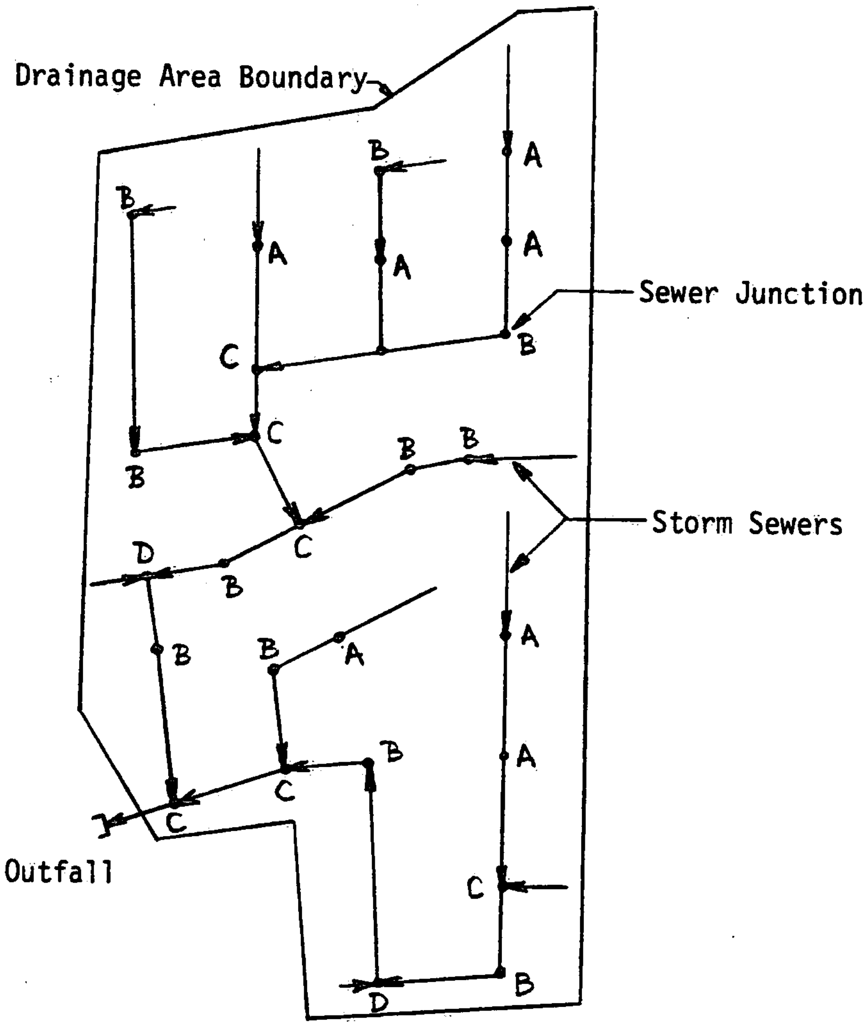
Author	Junction Characteristics				Reference
	Flow	Benching	(D_m/D_o)	(D_ℓ/D_o)	
Lindvall	P[1]	B2,B3	1.0	0.4-1.0	10
Marsalek	P,OC[2]	B1 - B3	1.0	0.5,1.0	13
Prins	OC	B1 - B3	1.0	0.67	15
Sangster et al.	P	B1	0.5-1.3	0.5-1.0	16

[1] Pressurized flow.
[2] Open-channel flow.

Table 8. Susceptibility of Selected Junctions to Sulphide Gas Releases

Junction Type	Susceptibility to Sulphide Releases According to Junction Benching		
	Low	Medium	High
90° Bend	B3	-	B1,B2
Main and Lateral	B3	B1,B2	B1
Two Opposed Laterals	B3	B2	B2

FIGURES



Legend

A - Two-pipe junction, no change in the alignment
B - Two-pipe junction with a bend
C - Junction of a main with a lateral
D - Junction of two opposed laterals

Fig.1. Storm Sewer System Layout With Various Types of Sewer Junctions

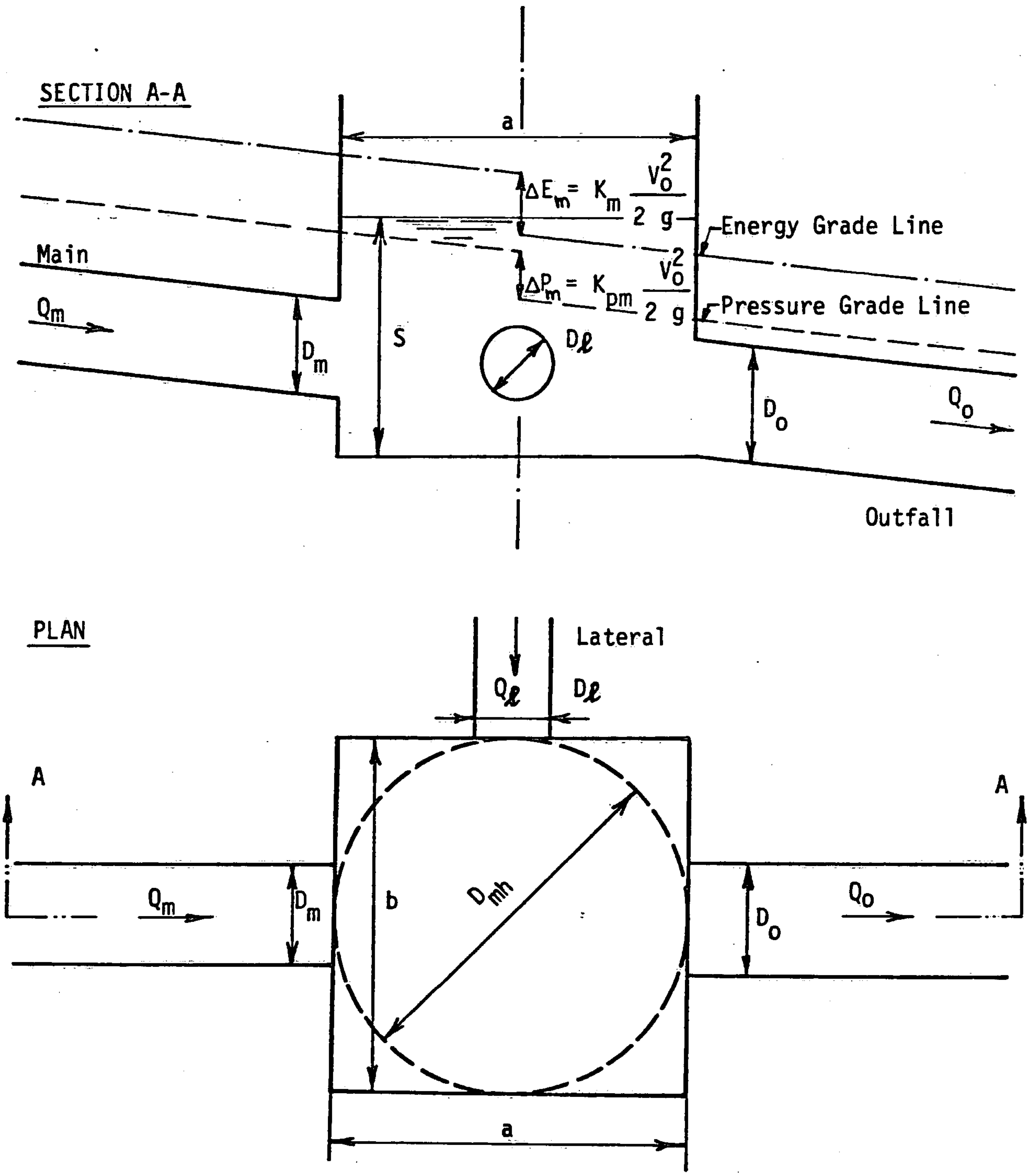
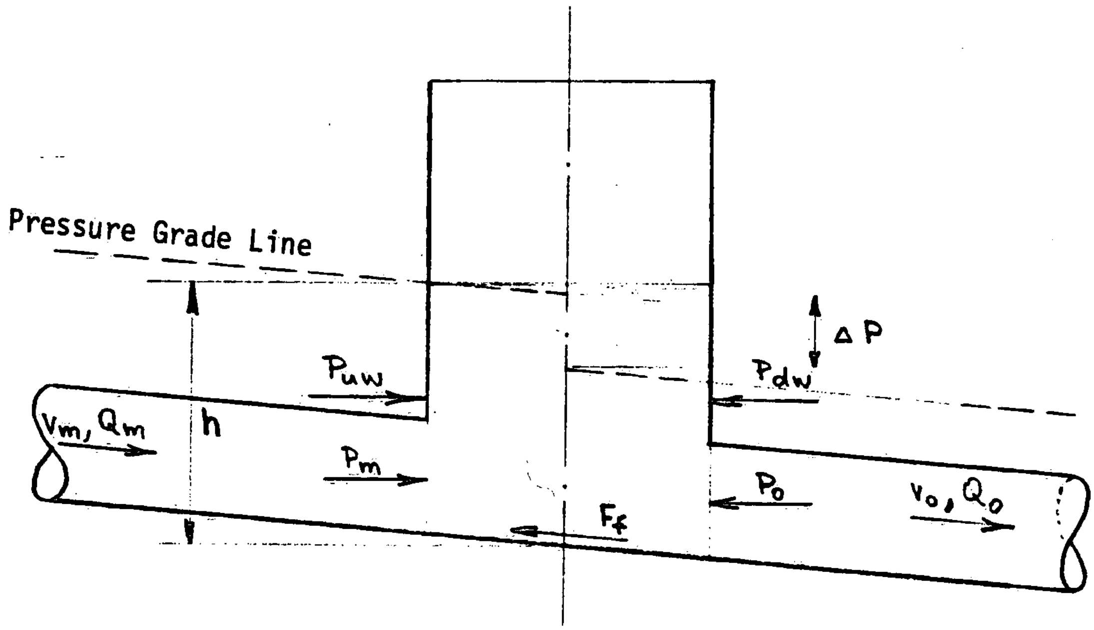


Fig.2. Junction of a Main and a Lateral: Flow Conditions and Notation

SECTION A-A



PLAN

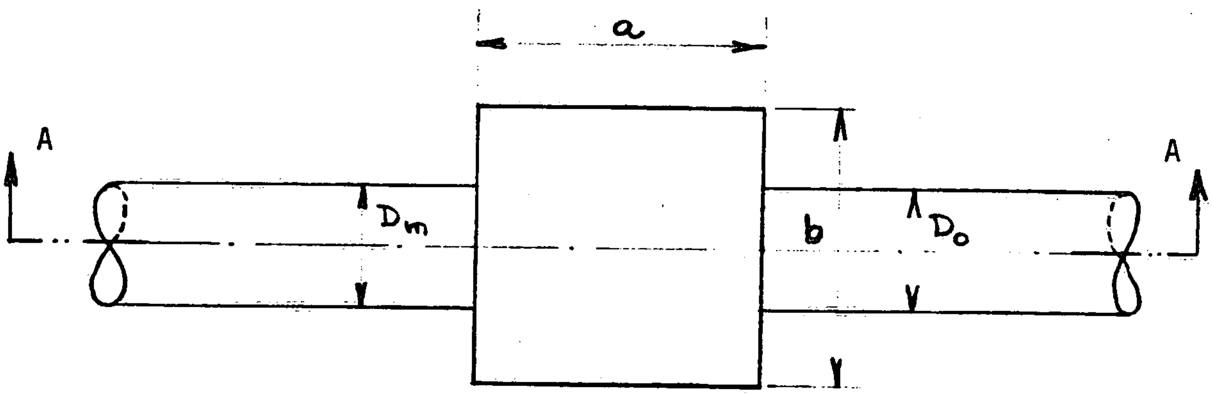


Fig.3. Application of the Momentum Equation to
a Straight-Flow-Through Junction

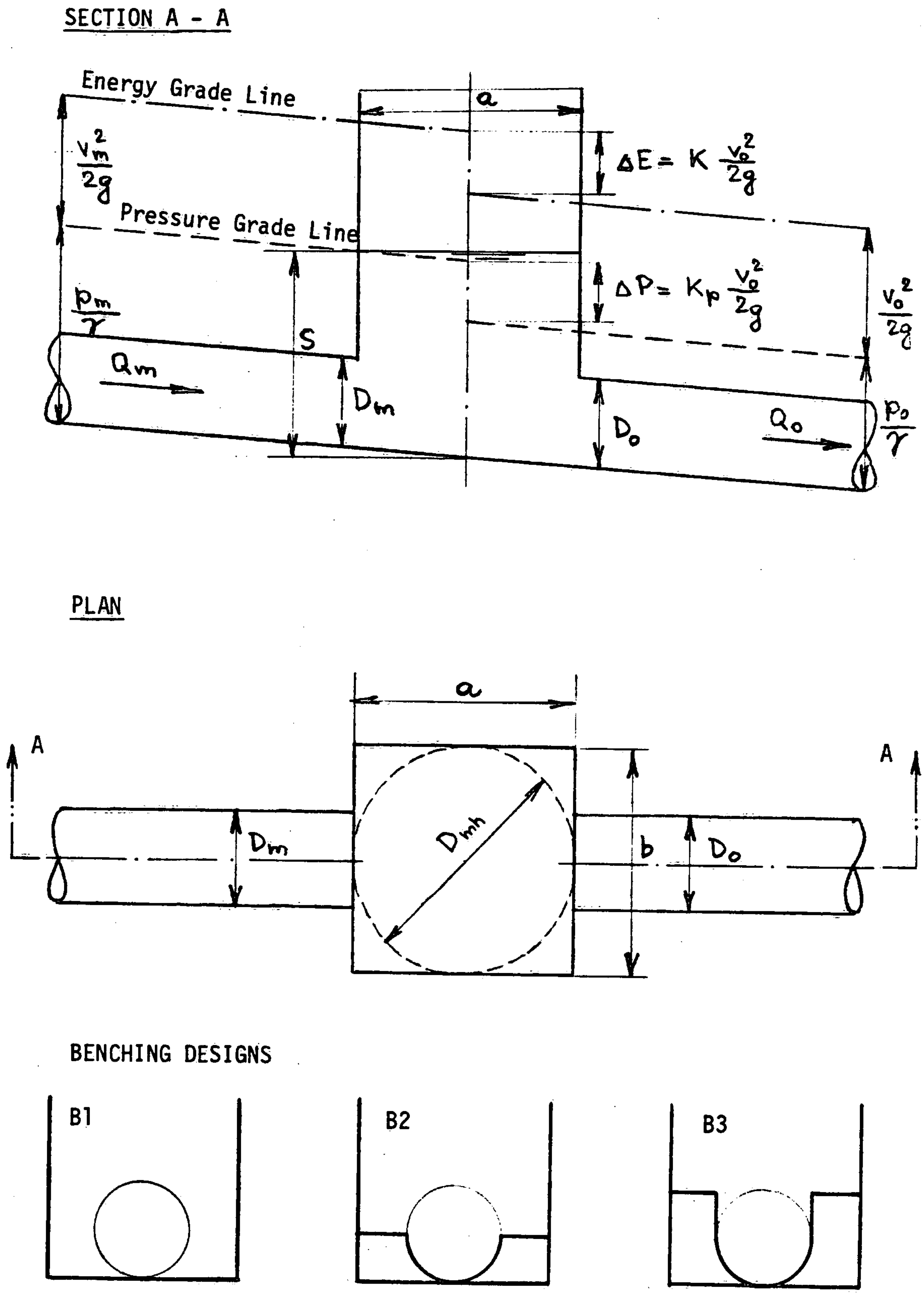


Fig.4. Straight-Flow-Through Junction

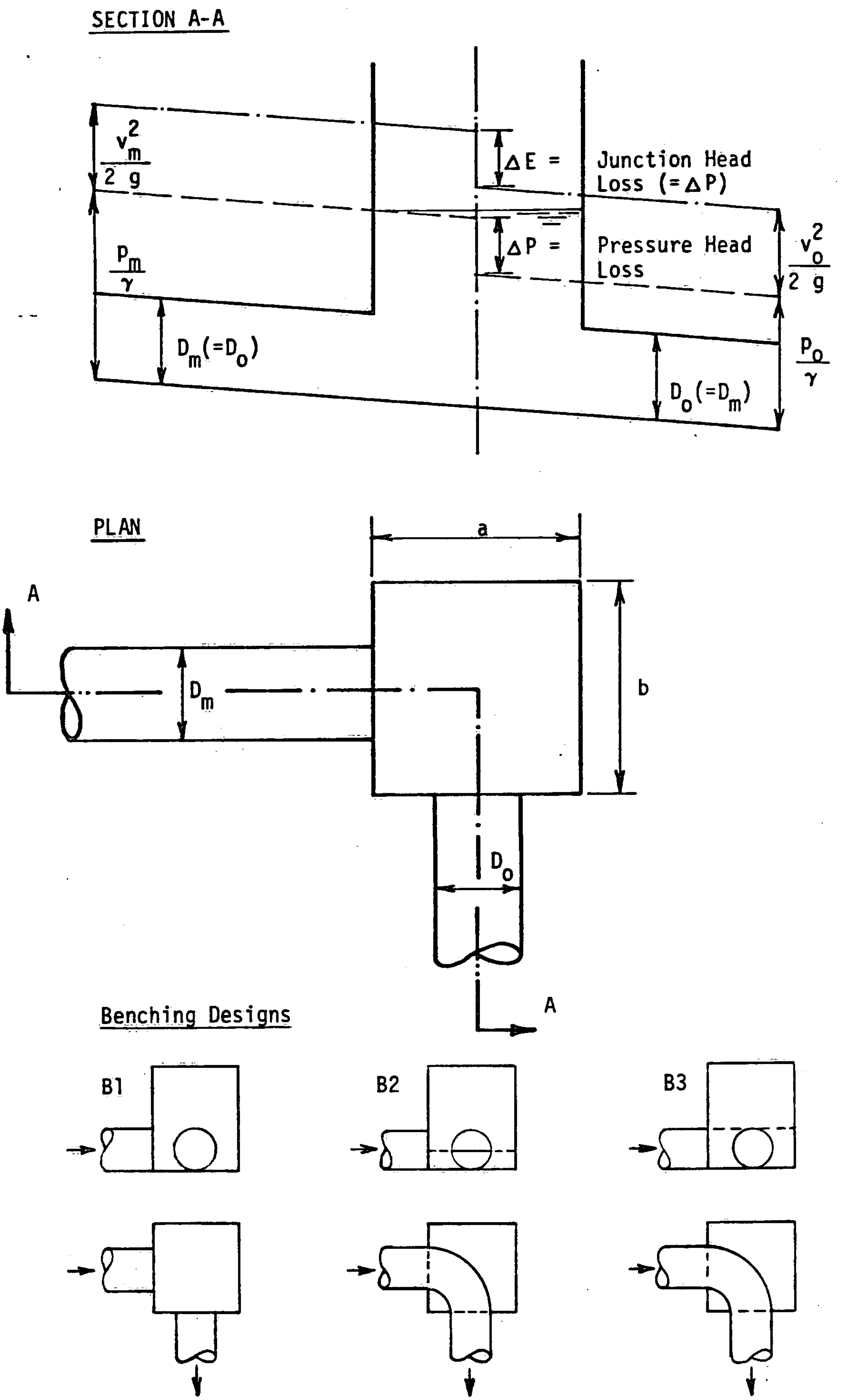
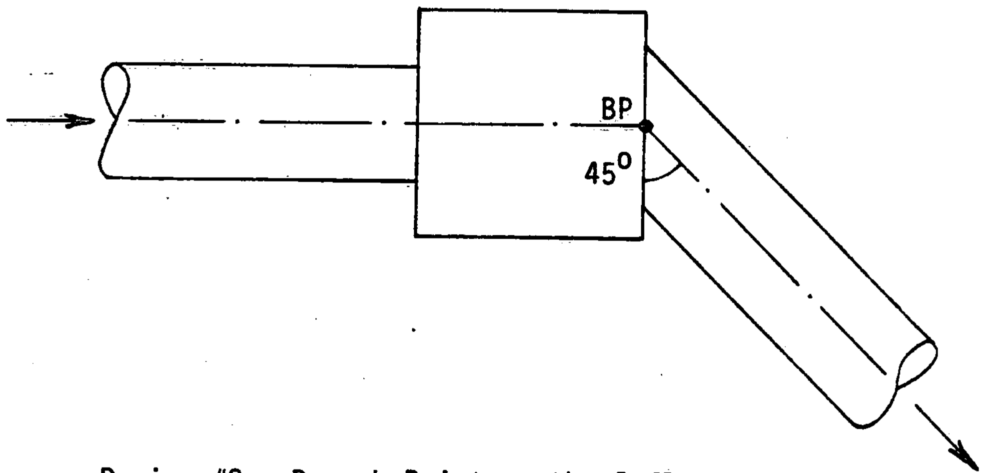


Fig.5. Manhole With a 90° Bend

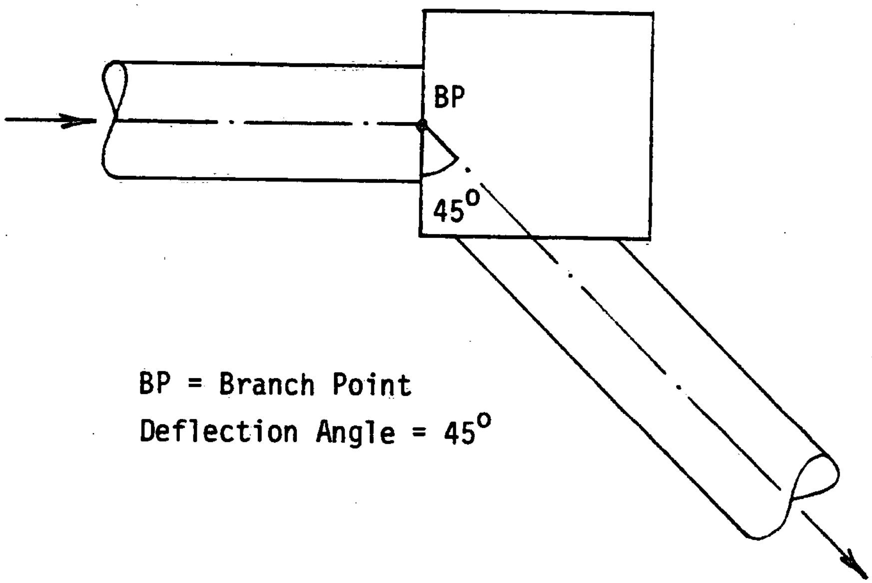
Design #1 - Branch Point on the Outfall Wall

$K_m = 0.60$



Design #2 - Branch Point on the Inflow Wall

$K_m = 2.20$



BP = Branch Point

Deflection Angle = 45°

Fig.6. Manhole Layouts With a 45° Bend and Different Locations of Branch Points (After ref.6)

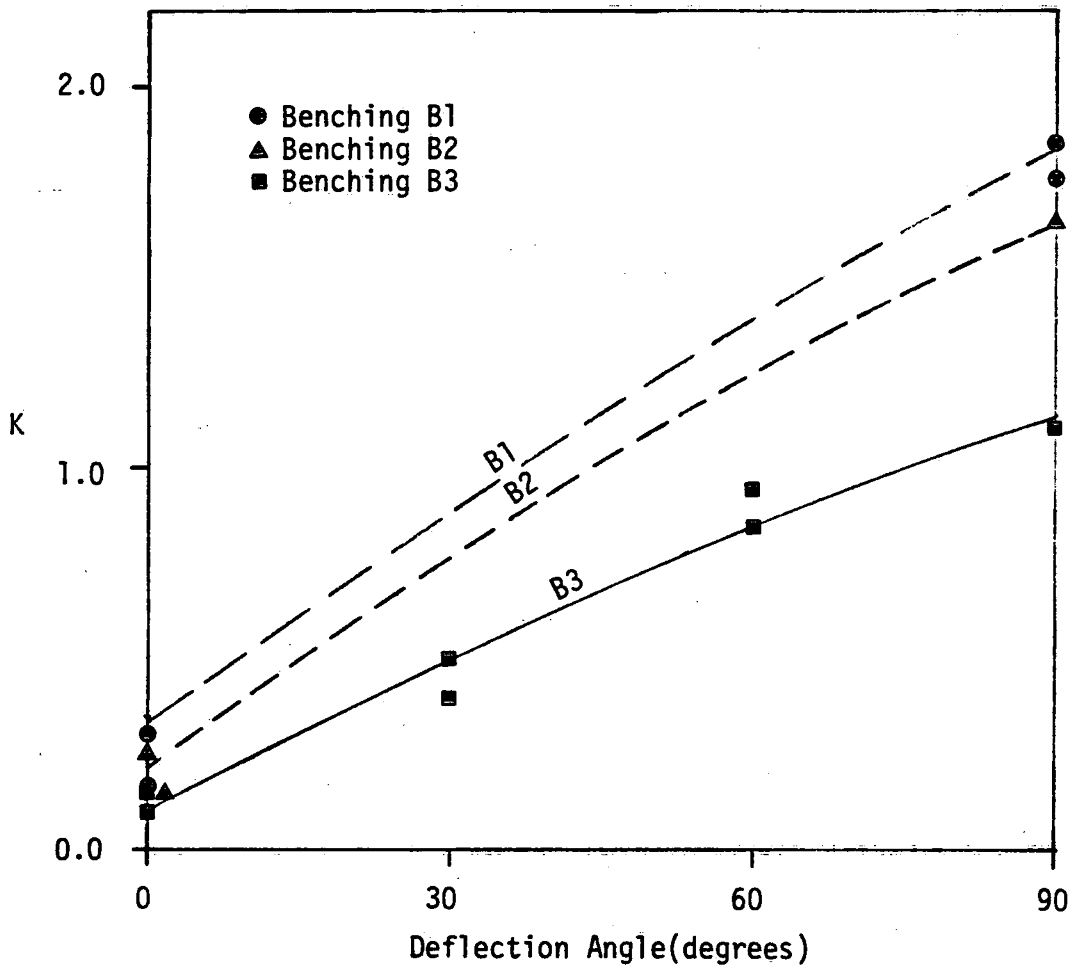


Fig.7. Head Loss Coefficients for Various Deflection Angles and Benchings

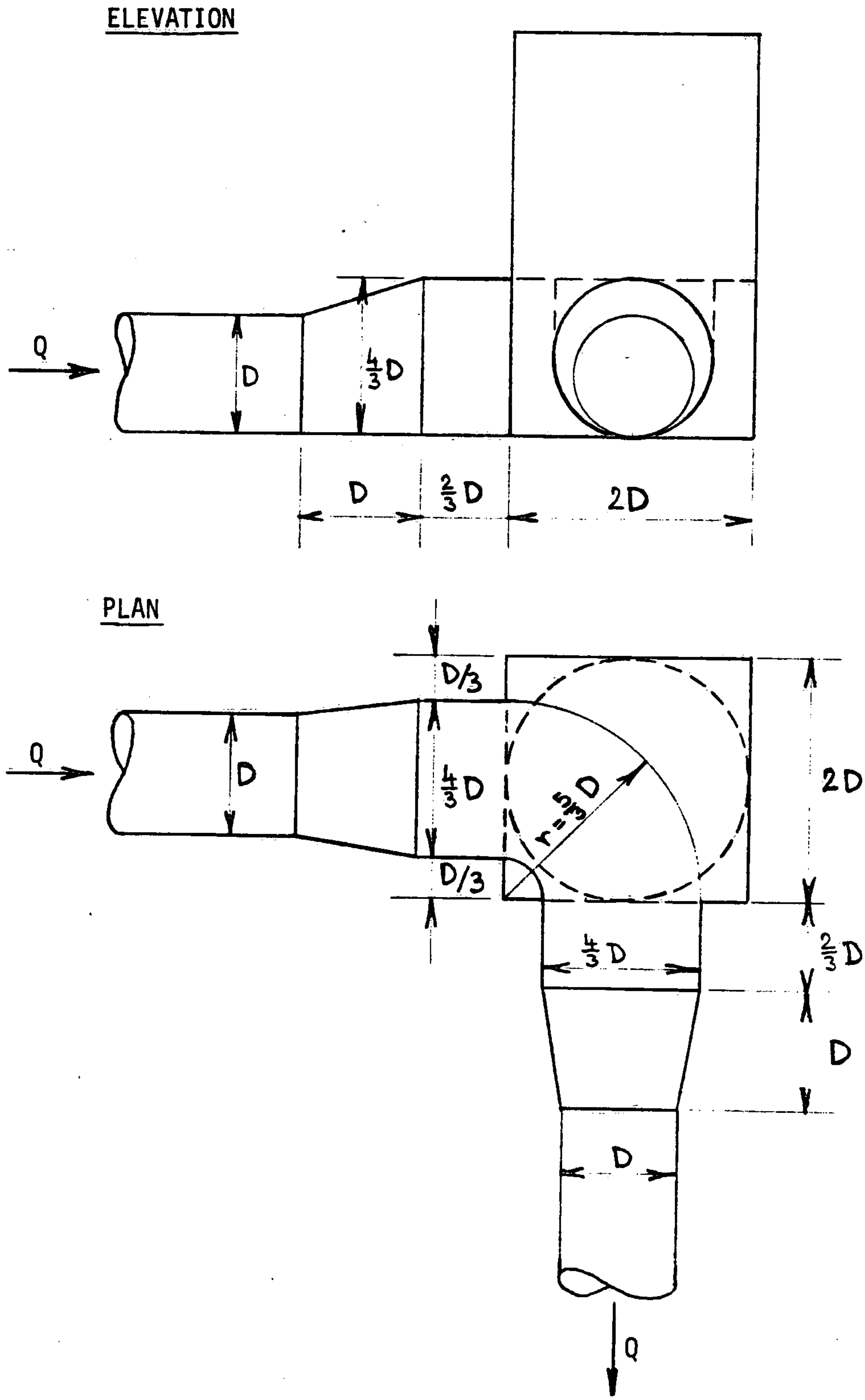


Fig.8. Low Head Loss Design for Manholes With a 90^{0} Bend(K = 0.65; after ref.13)

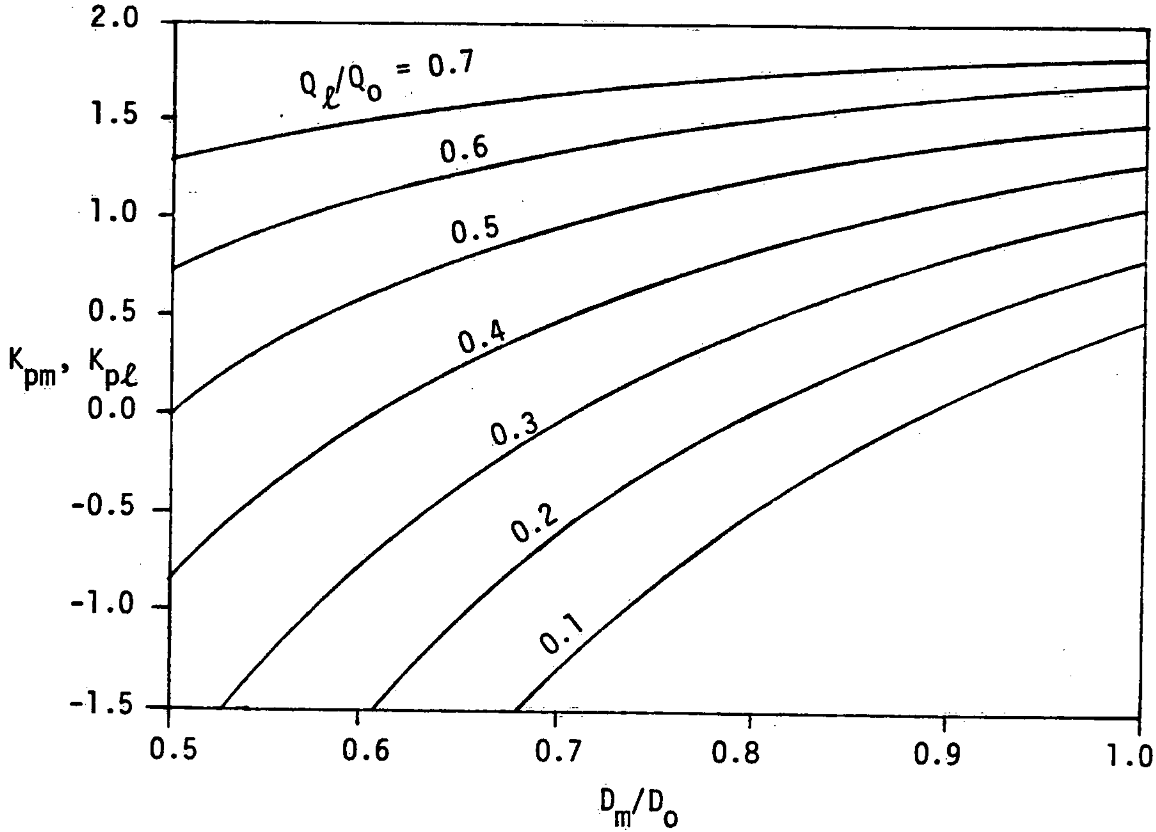
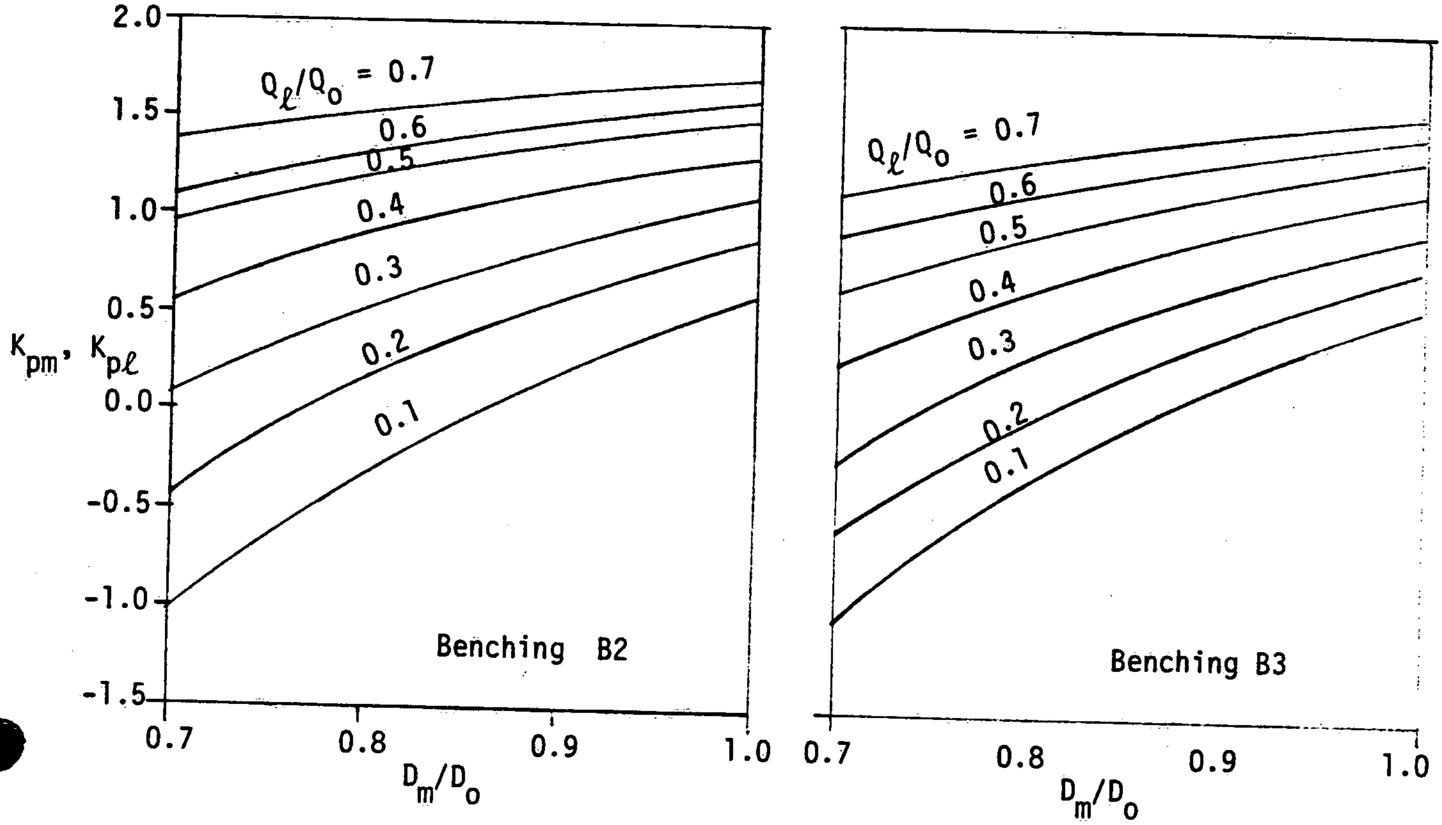


Fig.9. Pressure Change Coefficients for Junctions of a Main and a Lateral Without Benching($D_m \sim D_\ell$;after ref.16)



Benching Designs

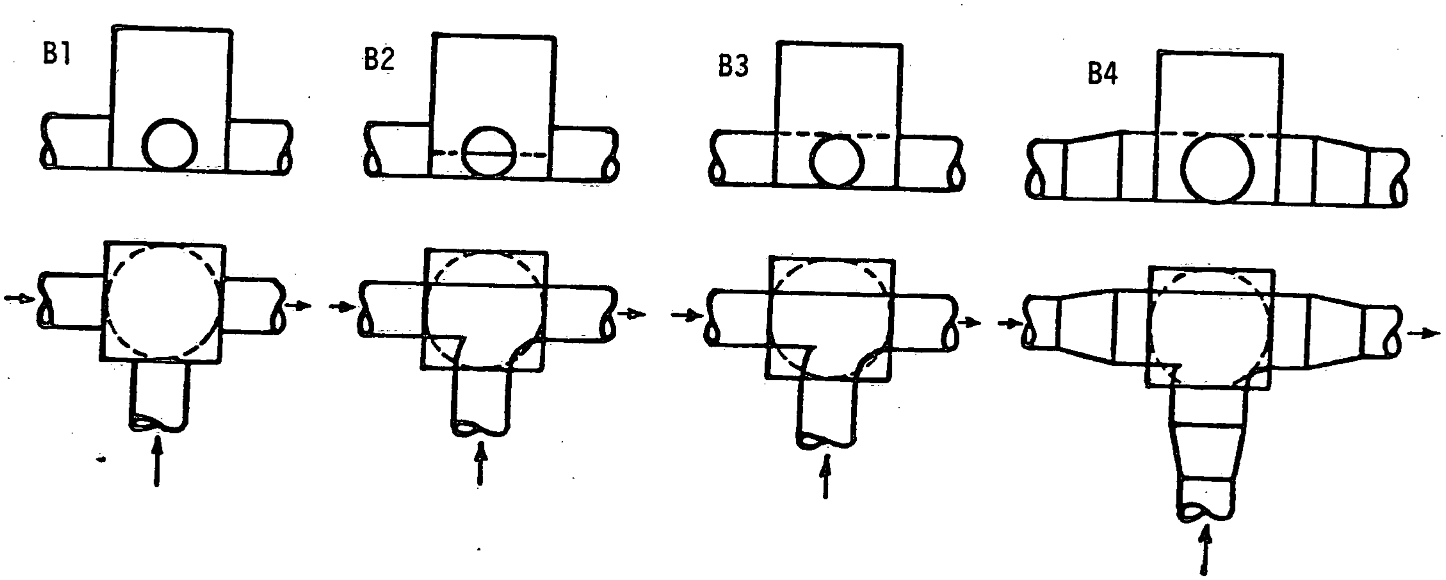


Fig.10. Pressure Change Coefficients for Junctions of a Main and a Lateral With Benchings ($D_m \sim D_\ell$; for $D_m/D_o<1$, all data extrapolated)

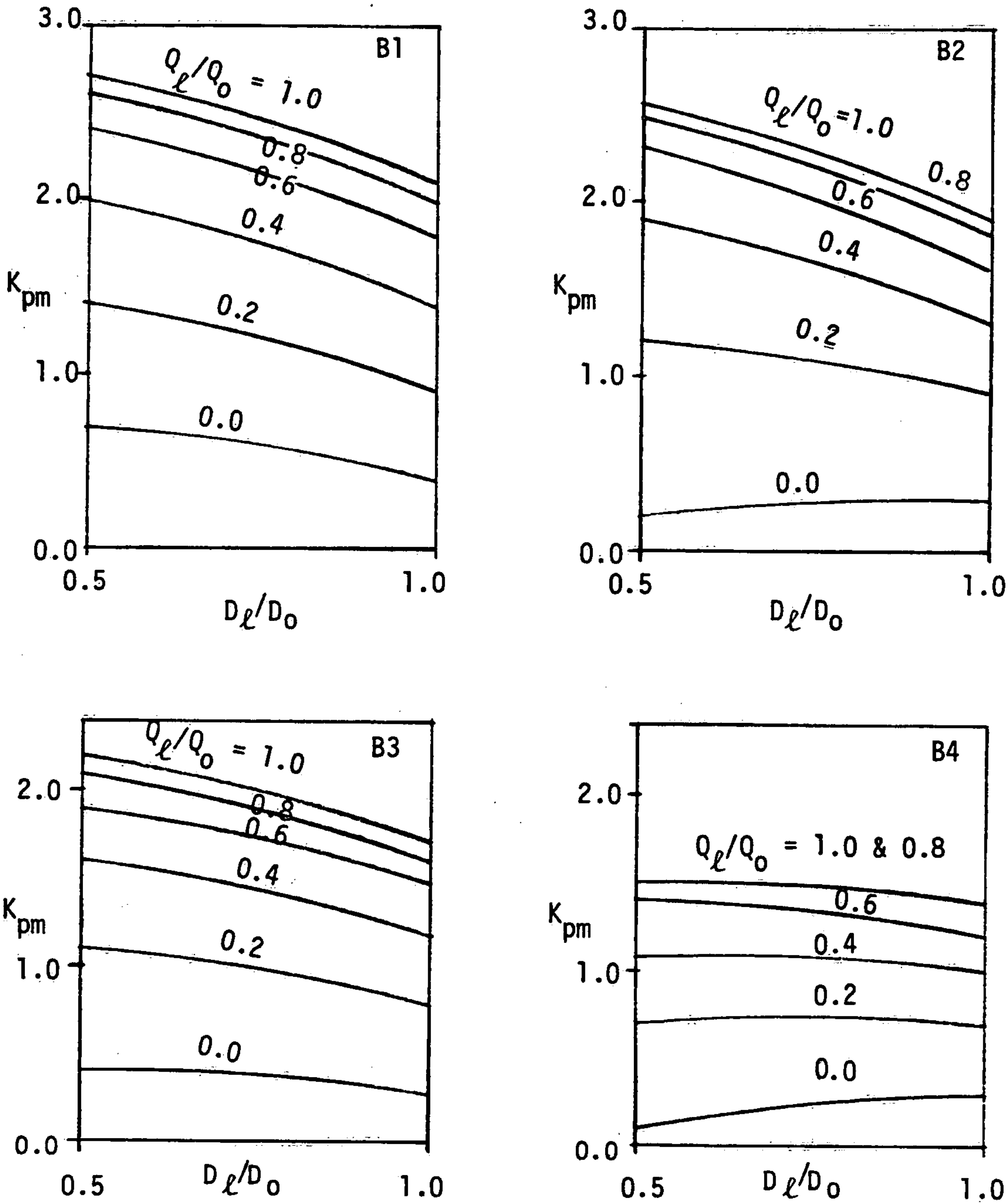


Fig.11. Main Pressure Change Coefficient for Junctions of a Main and a Lateral With Benchings(D_m=D_o; $D_\ell \leq D_o$; for $0.5<D_\ell/D_o<1$, data interpolated)

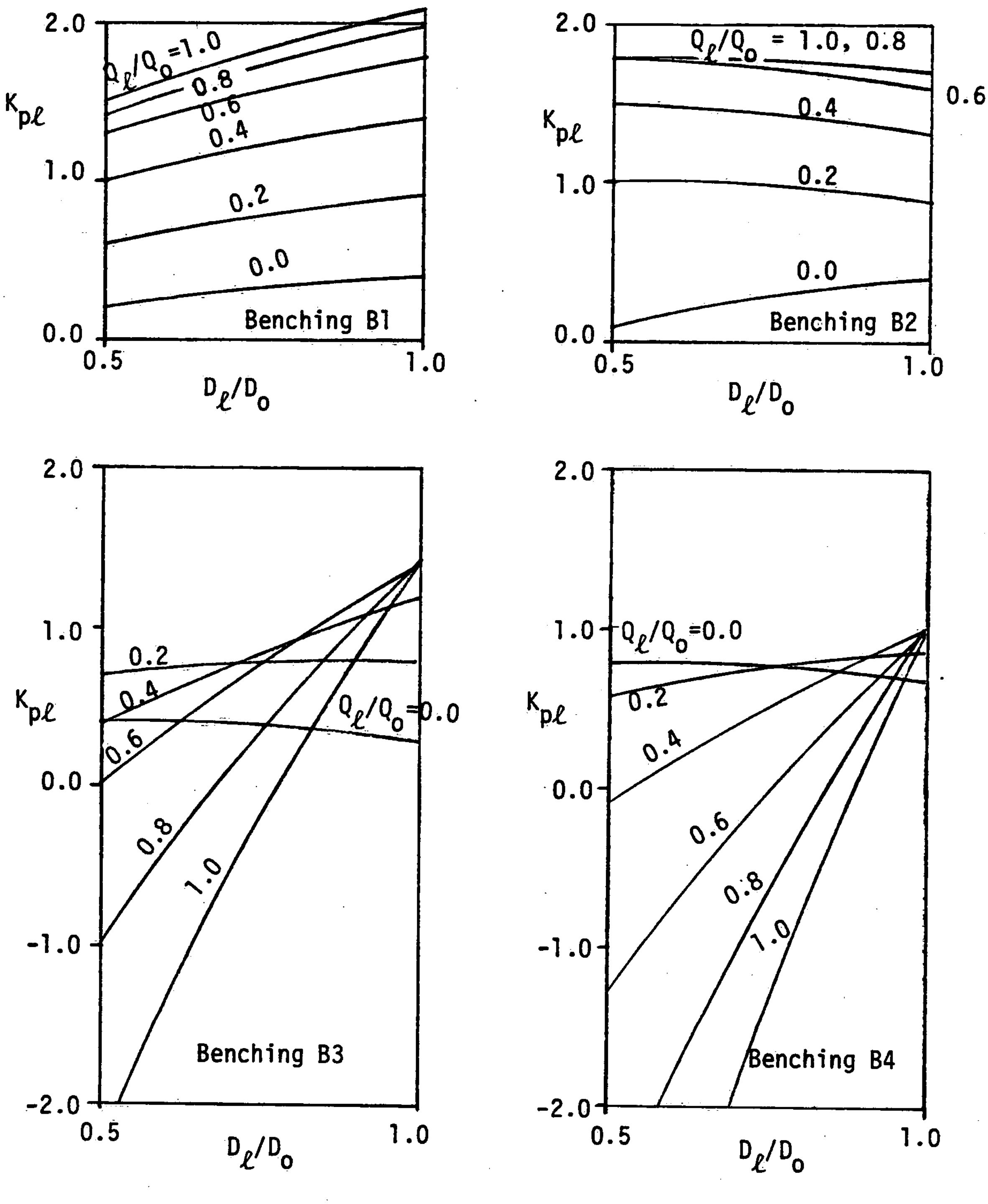


Fig.12. Lateral Pressure Change Coefficient for Junctions of a Main and a Lateral With Benchings ($D_m=D_o$; $D_\ell \leq D_o$; for $0.5<D_\ell/D_o<1$, data interpolated)

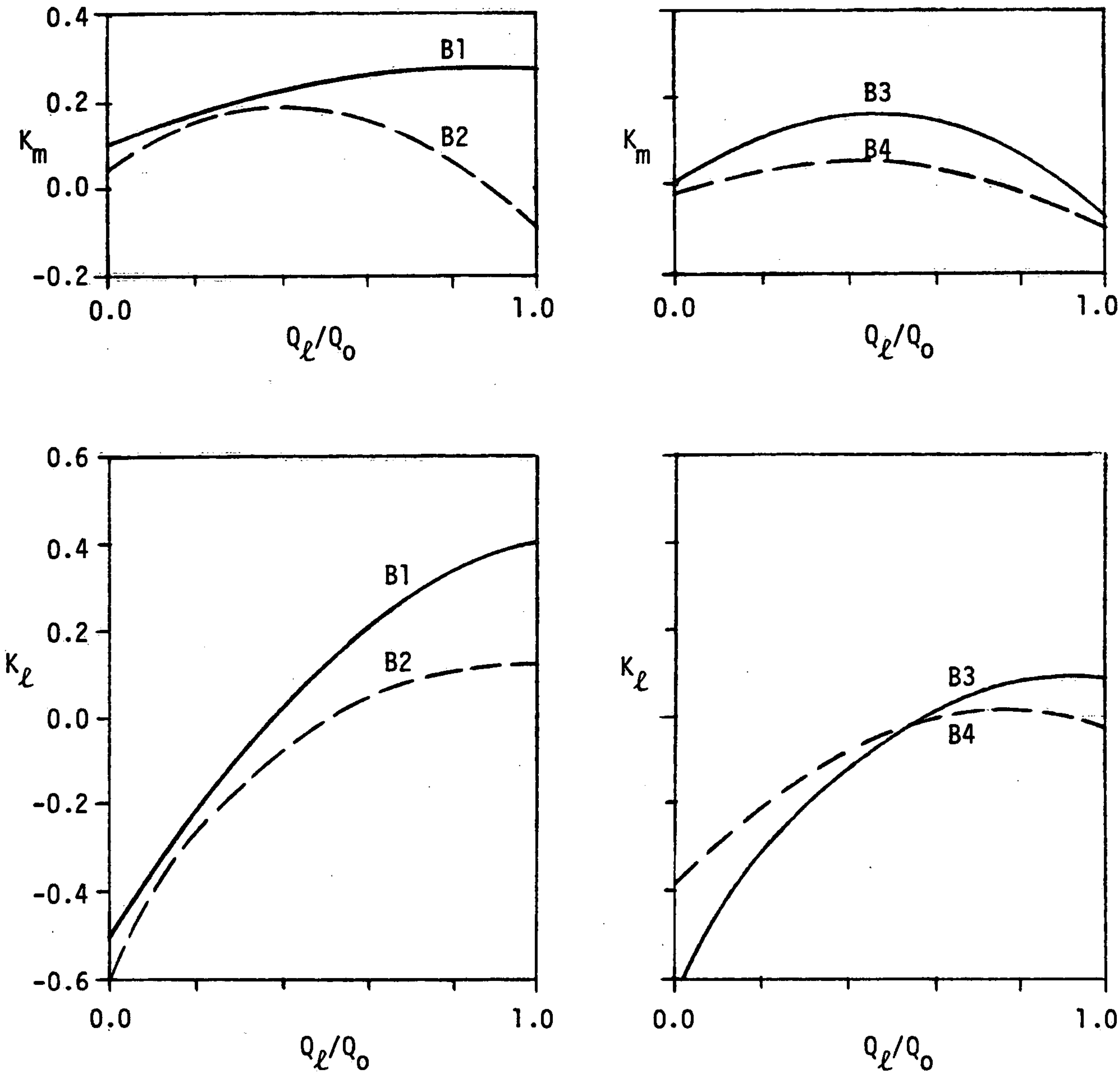


Fig.13. Junctions of a Main and a Lateral: Head Loss Coefficients for Subcritical Open-Channel Flow(after ref.13)

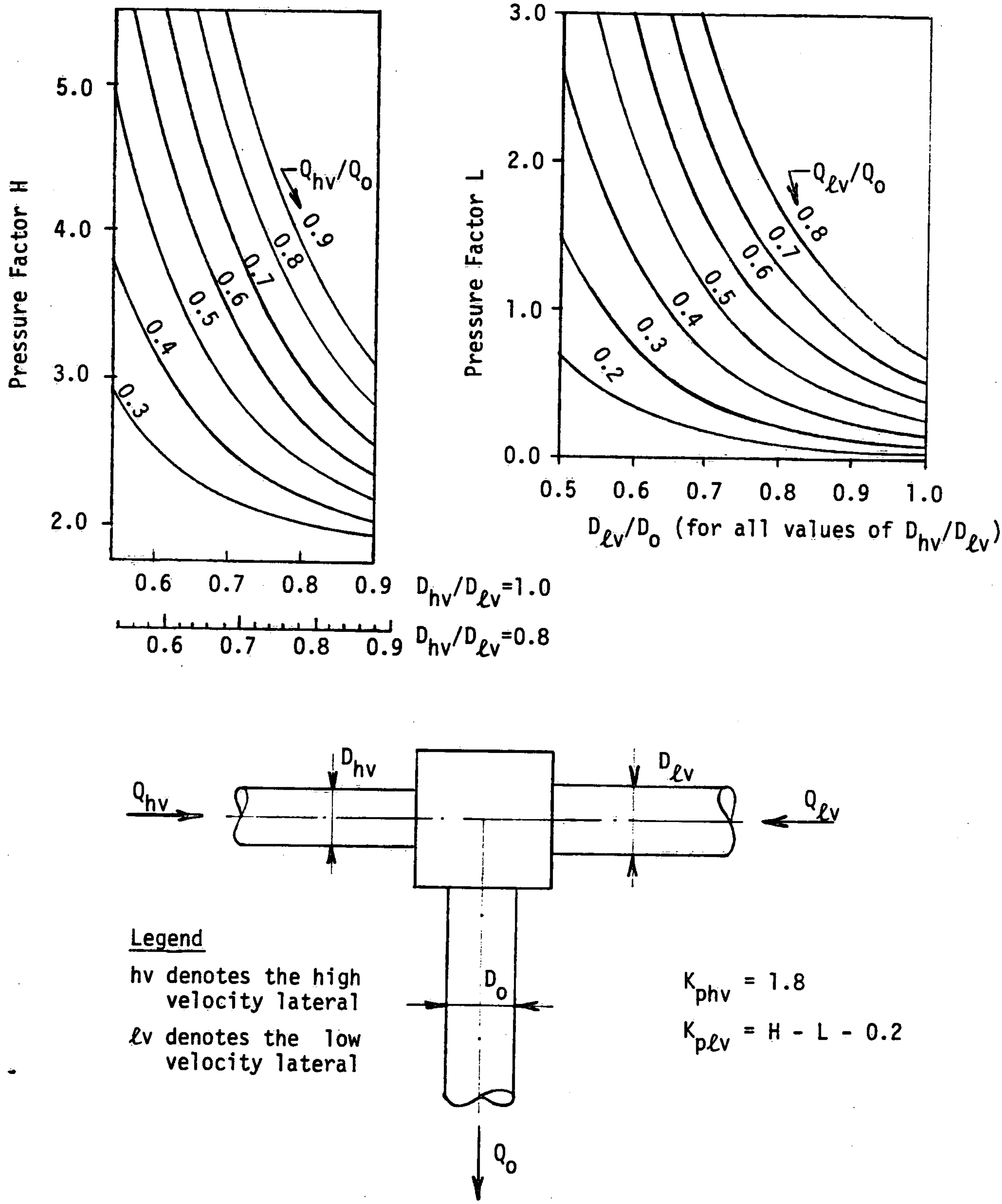
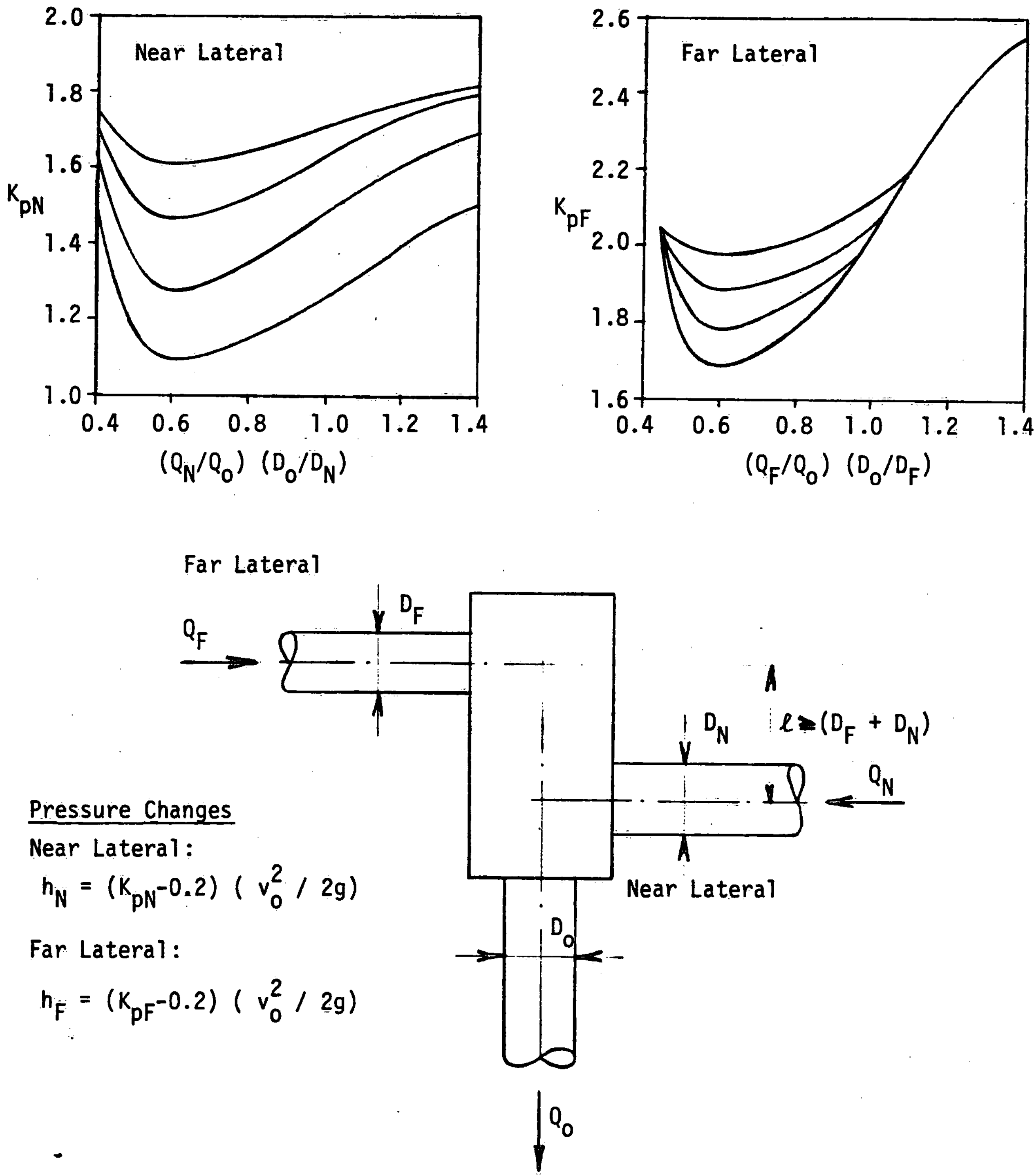


Fig.14. Junctions of Two In-Line Opposed Laterals: Pressure Change Coefficients(No benching; after ref.16)



Pressure Changes

Near Lateral:

$$h_N = (K_{pN}-0.2)\ (v_o^2 / 2g)$$

Far Lateral:

$$h_F = (K_{pF}-0.2)\ (v_o^2 / 2g)$$

Fig.15. Junctions of Two Offset Opposed Laterals: Pressure Change Coefficients(No benching; after ref.16)

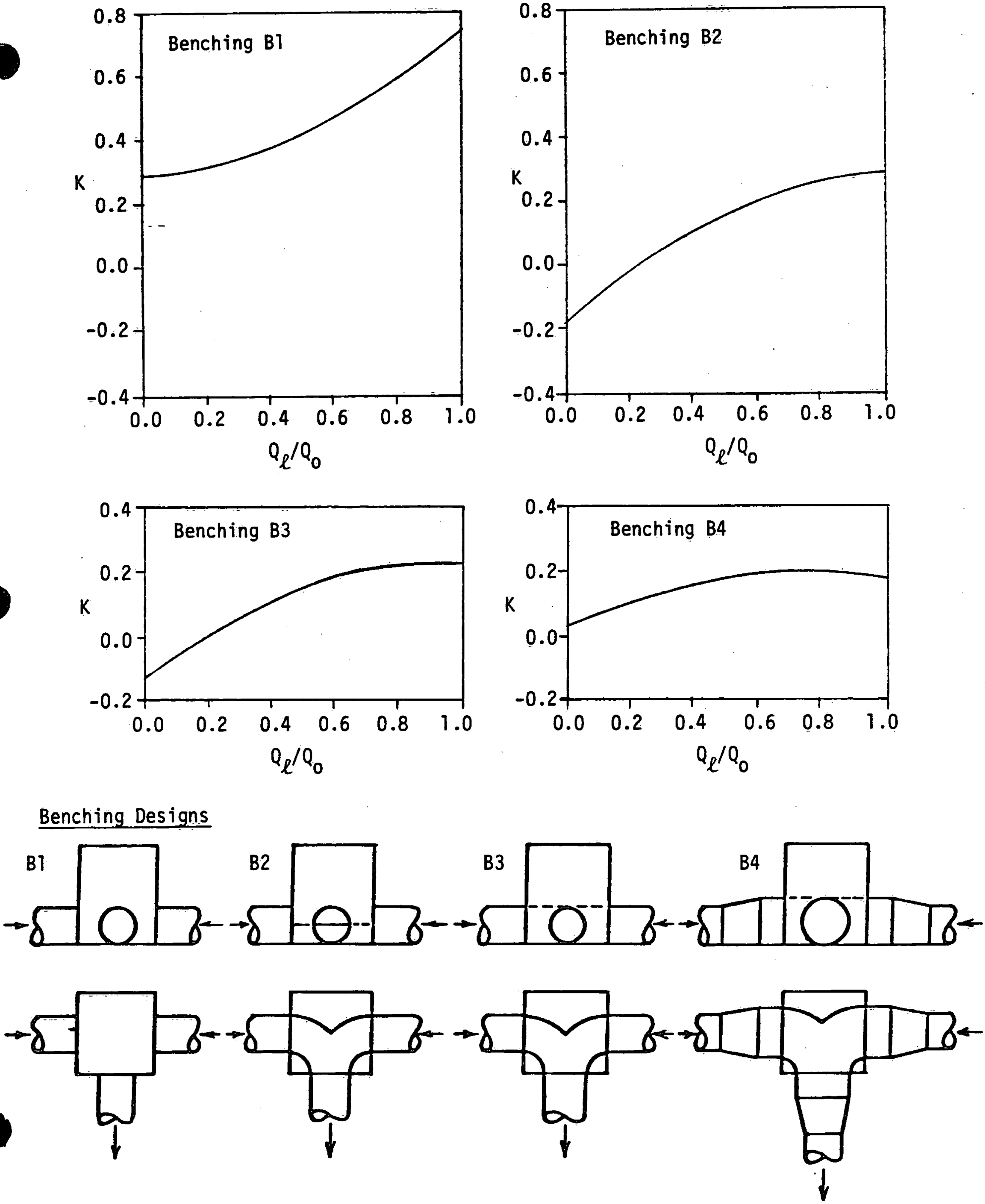


Fig.16. Junctions of Two In-Line Opposed Laterals: Head Loss Coefficients for Subcritical Open-Channel Flow(after ref.13)

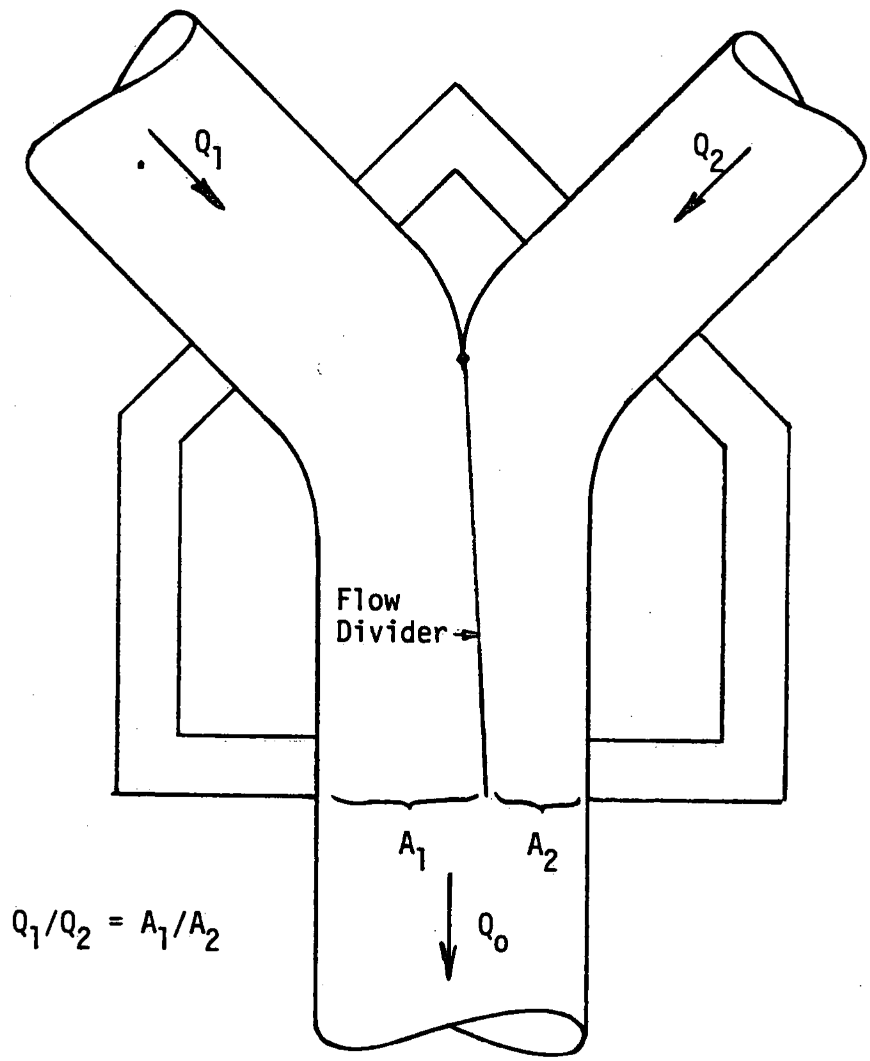
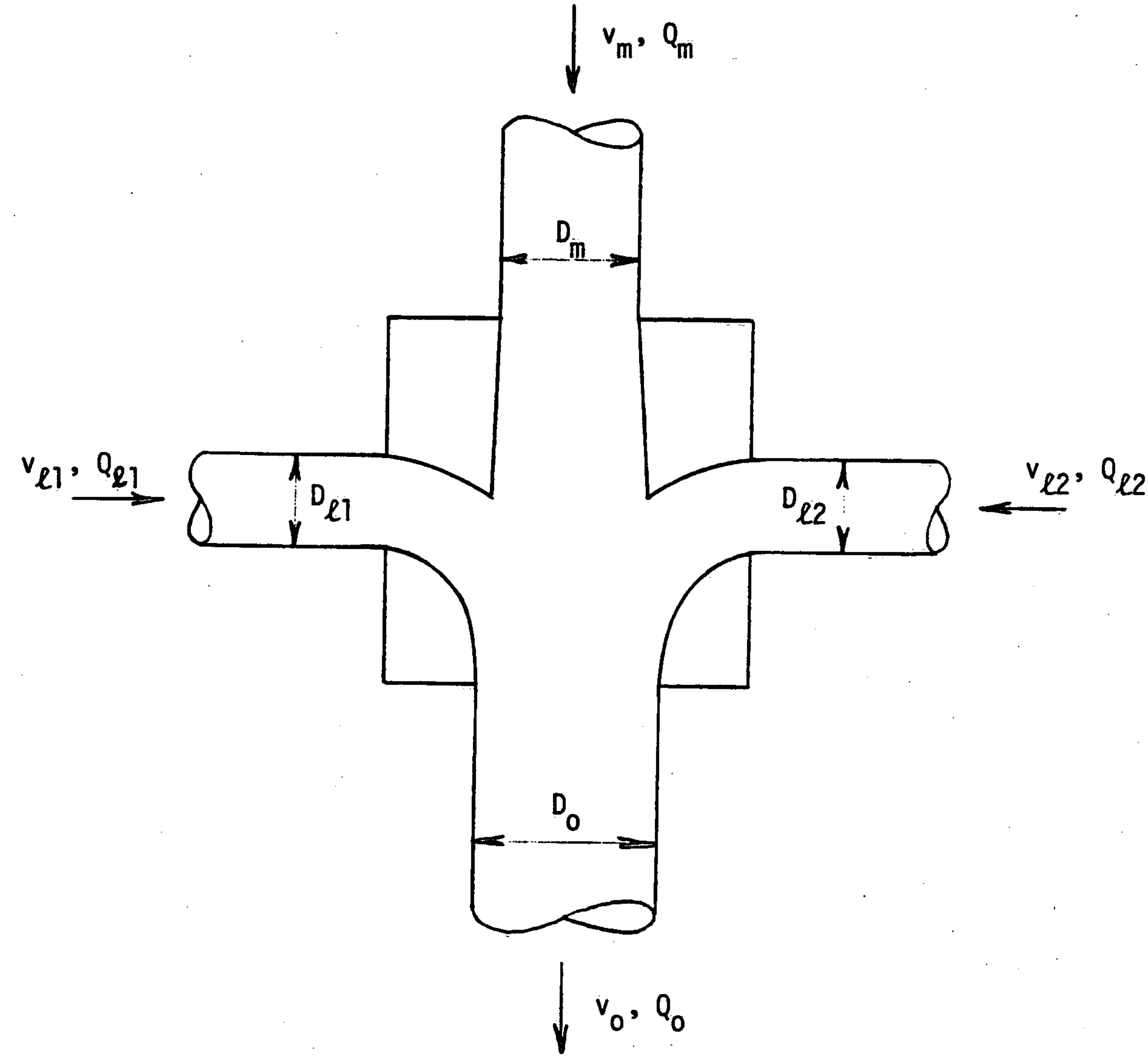


Fig.17. Y-Junction With a Flow Divider(after ref.15)



Simplifications of Four-Pipe Junctions

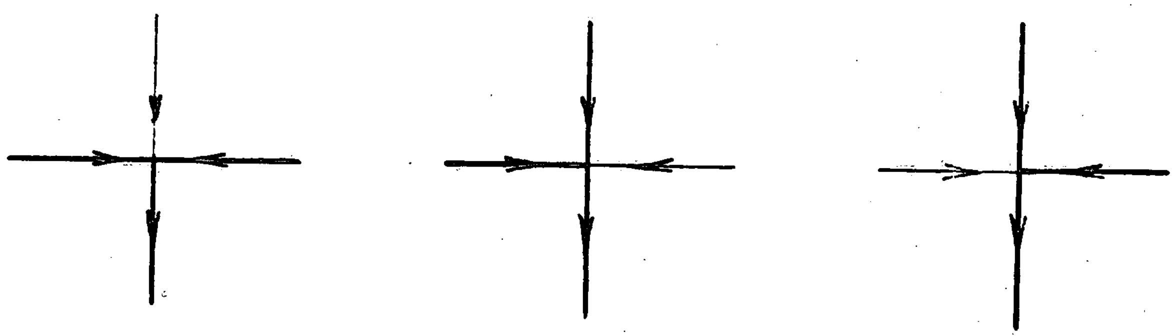


Fig.18. Four-Pipe Junctions