HYDRAULIC MODEL STUDY OF A MANHOLE JUNCTION

by

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Table of Contents

List of Tables and Figures

Introduction

Manholes are structures placed in sewer systems to allow workmen to descend to the pipelines for the purpose of cleaning and inspection. The bottom of these structures usually consists of a four foot circular concrete slab containing U-shaped channels which convey the flow of sewage while it passes through the manhole. Present sewer construction practices require the placement of manholes at changes in direction, changes in pipe size, large changes in grade, and intervals of between 300 and 500 feet if other access is not provided. These requirements make manholes the most common appurtenance in sewer systems.

Because manholes are located at bends, contractions, expansions, and junctions, they are invariably sites of hydraulic transitions. These transitions result in energy losses causing a depression of the energy grade line and a possible change in water surface elevation. Generally these energy losses are small, and relatively unimportant, in comparison with the frictional losses incurred in the pipe lines. However, under certain conditions, possibly involving maximum flow and high velocities, the transitional losses can become quite large and can exert an important influence on the capacity of the system.

The purpose of proper manhole design is to minimize the energy losses by providing smooth transitions and to compensate for the losses which do occur by lowering the invert elevation of the outlet pipes. Estimating the proper drop in invert elevation is a very important aspect of design because an under estimation of the drop could conceivably cause unsteady slug flow to occur with a resulting reduction in capacity of the system, or surcharging of the inlet pipes resulting in flooded upstream connections. An overestimation of the drop will result in increased construction costs as a result of excessive excavation, increased pipe sizes, or increased pumping costs.

The present practice in manhole design of sanitary sewers is to apply empirical allowances for loss of head with little regard to the size of flow or velocities involved. Because of the complexity of the energy losses involved in transitions of this type, a purely rational analysis is not possible at this time; however, an analysis based on the conservation of momentum has been suggested as an approach to the problem.

The object of this research is the experimental investigation of the hydraulics of a flow-through manhole with one lateral connection. An attempt will be made to use the conservation of momentum principle to develop relationships which will describe the experimental values obtained in the model study.

Literature Review

Relatively little work has been done to develop methods for predicting the transitional losses at open channel manhole junctions of the type found in sanitary sewer systems. In most cases, engineers have relied on "rule of thumb" procedures for the design of this type of structure.

Fair and Geyer (5) list some examples of design procedures used to determine the drop in sewer invert elevations in manholes. For example:

1. $h_1 = \frac{1}{2}(d_2 - d_1)$ for pipes smaller than 24 inches,

   $h_1 = 3/4\,(d_2 - d_1)$ for pipes 24 inches and larger, where

   h_1 = drop in invert elevation

   d_1 = depth of flow in upstream pipe

   d_2 = depth of flow in downstream pipe.

2. Keep the 0.8 d line continuous for all pipes on the principle that it represents the line of maximum velocity.
3. Base flow calculations on a roughness factor greater than that used in straight line runs; for example: N = 0.015 instead of 0.012.
4. Allow for a drop of 0.1 feet in a through manhole; 0.2 feet in the presence of one lateral or bend; and 0.3 feet for two laterals.

Steel (13) suggests providing a drop of 0.04 feet in invert elevation for a change in direction without a change in pipe size. For sewers with a change in size, he advocates placing the crowns of all the pipes at the same elevation.

Camp (2) stated that the practice of designing transitions by placing the crowns at the same elevations without regard to the energy conditions is faulty. He maintained that a transitional head loss must be applied to the energy grade line. By assuming that all changes due to transitional losses take place at the center of the transition (Figure 1, page 8) he obtained the equation

$$h_2 + h_e = h_1 + h_i$$

where

h_1 and h_2 = the height of the energy grade line above pipe invert in pipes one and two.

h_e = energy loss in transition

h_i = required drop in invert elevation.

Now, assuming that the energy loss is proportional to the difference in velocity heads

$$h_e = K\ (h_{v2} - h_{v1})$$

where

h_{v1} and h_{v2} = velocity head in pipes 1 and 2

K = proportionality constant

and combining the two equations, the required drop in invert elevation becomes

$$h_i = (h_2 - h_1) + K\ (h_{v2} - h_{v1}).$$

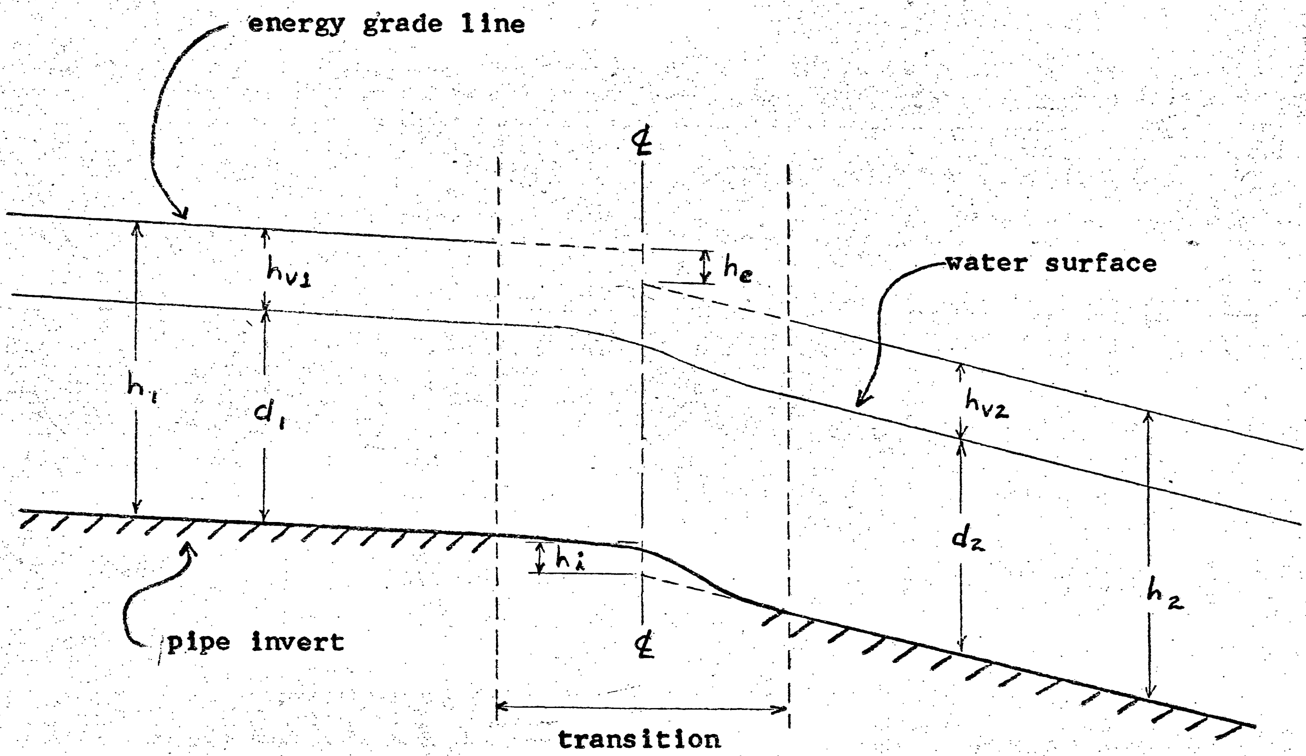


Figure 1: Camp's approach to analysis of transitions (2)

Hinds (8) investigated proportionality constants for flume and siphon transitions. He suggested that K may have a value as low as 0.1 for increasing velocities and as low as 0.2 for decreasing velocities provided the flow is subcritical. Camp suggested a minimum allowable head loss of 0.02 feet for a straight through manhole transition and a larger minimum if a curve is included. He also pointed out that the value of h_i may be negative, which would indicate a rise in invert elevation. This rise should be avoided, by assuming h_i is equal to zero because the resulting depression formed would be a possible location of grit deposition.

Camp did not present an analysis of junctions of sewers in manholes, but he did make some observations about the problem.

1. An additional allowance in head should be provided for both inlet paths to compensate for the impact losses due to the converging streams.
2. The impact loss at a junction is, theoretically, subject to consideration by the momentum principle.
3. It is not practicable to apply the momentum theory to the actual design of junctions because the pressure along the walls and floor cannot be accurately determined because of the influence of the impact of the streams and the curvature of the walls.

Some theoretical and experimental work has been done on the losses at pipe intersections. Stevens (14) developed a theoretical analysis for the losses in a junction of any number of pipes flowing full and under pressure. By stating that all of the pipes intersected at the same point, he assumed that the pressure at the junction for all pipes would be equal. By writing an expression using the momentum equation

$$J - P = 2H - 2h_1 \frac{a_1}{A} \cos \theta_1 - 2\,h_2 \frac{a_2}{A} \cos \theta_2 - \dots - 2h_n \frac{a_n}{A} \cos \theta_n$$

and combining it with an expression for the energy head loss

$$i = J - P - H + \frac{q_1}{Q} h_1 + \frac{q_2}{Q} h_2 + \dots + \frac{q_n}{Q} h_n$$

Stevens obtained a general expression for evaluating the energy loss in intersecting pipes

$$i = H + h_1 \left(\frac{q_1}{Q} - 2 \frac{a_1}{A} \cos \theta_1\right) + h_2 \left(\frac{q_2}{Q} - 2 \frac{a_2}{A} \cos \theta_2\right) + \dots +$$

$$h_n \left(\frac{q_n}{Q} - 2 \frac{a_n}{A} \cos \theta_n\right)$$

where

J = pressure at junction

P = maximum average pressure in main below junction

H = velocity head in main

A = area of main

Q = flow in main

$h_1, h_2, \dots h_n$ = velocity head in respective branches

$a_1, a_2, \dots a_n$ = area of respective branches

$q_1, q_2, \dots q_n$ = flow in respective branches

$\theta_1, \theta_2, \dots \theta_n$ = angles respective branches make with main

i = energy lost by impact and eddies expressed as

Mason and Blaisdell (9) conducted an experimental investigation to determine the losses at draintile junctions. The experimental equipment consisted of straight-through lines with laterals of the same size flowing full and entering at angles varying between 15 and 165 degrees, with flow velocities between 2 and 15 feet per second. No theoretical analysis of the problem was attempted; however, an energy loss coefficient, K, for each junction was plotted against the ratio of the flow in the branch to the flow in the downstream main $\frac{Q_b}{Q_d}$. The loss in energy head for a similar situation can be obtained by multiplying the K value, determined from the curves, by the velocity head of the downstream main.

Sangster, Wood, Smerdon, and Bassy (11) presented an analytical and experimental investigation of the effects of open junctions on the magnitude of pressure changes in closed conduits flowing full. The configuration under study was a straight main line with a 90 degree lateral entering in a junction box (Figure 2). The theoretical analysis was an application of the momentum equation with the assumption that the 90 degree lateral contributed no momentum in the direction of the main line. With this assumption the equation for the head loss through the main line was found to be:

$$h_2 = 2 \frac{V^2}{2g} \left[1 - \left(\frac{Q_1}{Q2}\right)^2 \left(\frac{A_1}{A_2}\right) \right]$$

The equation was then solved for a pressure change coefficient K, which is equal to the change in pressure over the velocity head

$$K_2 = 2 \left[1 - \left(\frac{Q_2}{Q_1}\right)^2 \left(\frac{D_1}{D_2}\right)^2 \right].$$

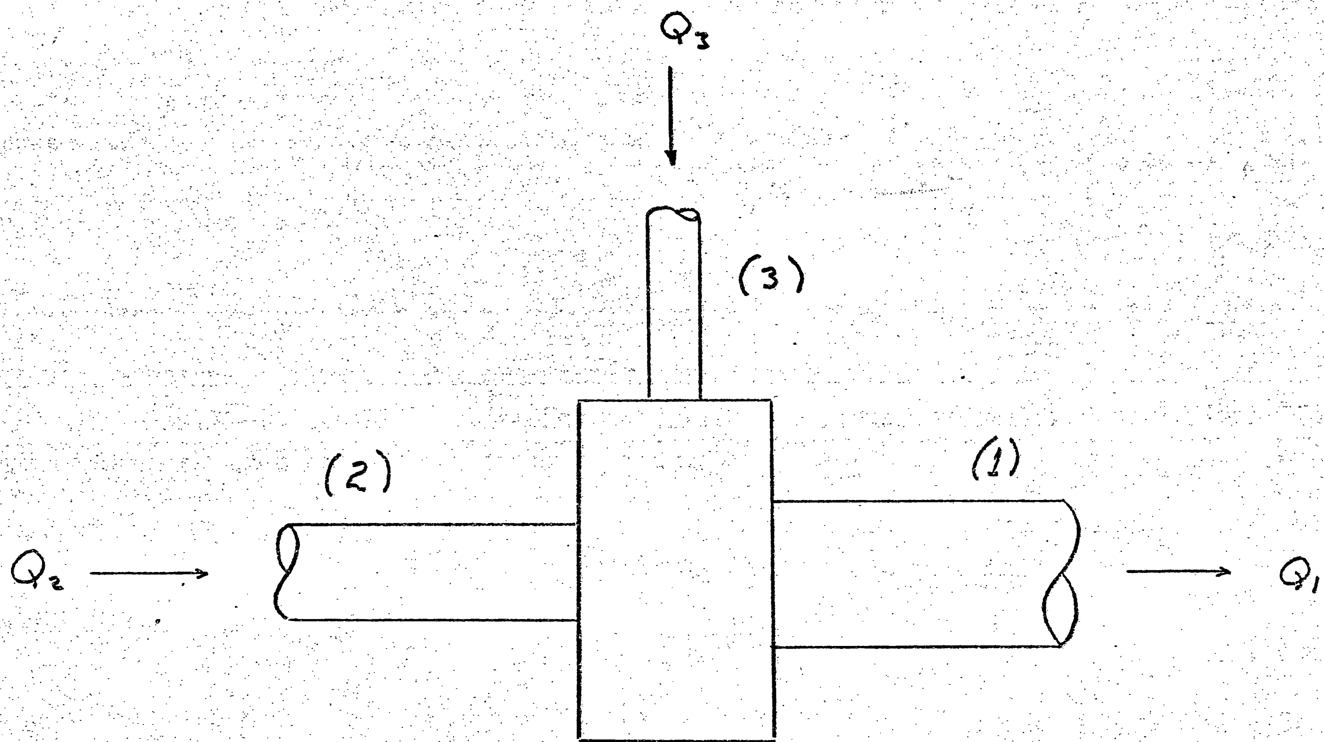


Figure 2: Sangster's junction box (11)

It was found that this equation agreed satisfactorily with the experimental results so long as less than 40 per cent of the total flow entered from the lateral. For the case where the major portion of the flow entered from the lateral, the equation was modified to the form:

$$K_2 = K_2' \left[1 - \left(\frac{Q_2}{Q_1}\right)^2 \left(\frac{D_1}{D}\right)^2 \right]$$

where K_2' is an empirically determined value used to compensate for the momentum effect contributed by the lateral flow.

Taylor (15) investigated the problem of combining and dividing subcritical flow in rectangular open channel junctions. The analysis was confined to one basic channel junction (Figure 3, page 14). The characteristics of the junction were:

1. Channels of equal width.
2. Flat bottom slopes.
3. Flow from channel 1 and 2 into channel 3.
4. Channels 1 and 3 lie in straight line.

The assumptions made for the analysis were:

1. Flow is parallel to the channel walls immediately above and below the junction.
2. Wall friction is negligible in comparison with other forces.
3. The depth in channels 1 and 2 are equal immediately above the junction.

By application of the momentum equation, with the further assumption that the depth remains constant in channel 2 within the transition, the equation was found to be:

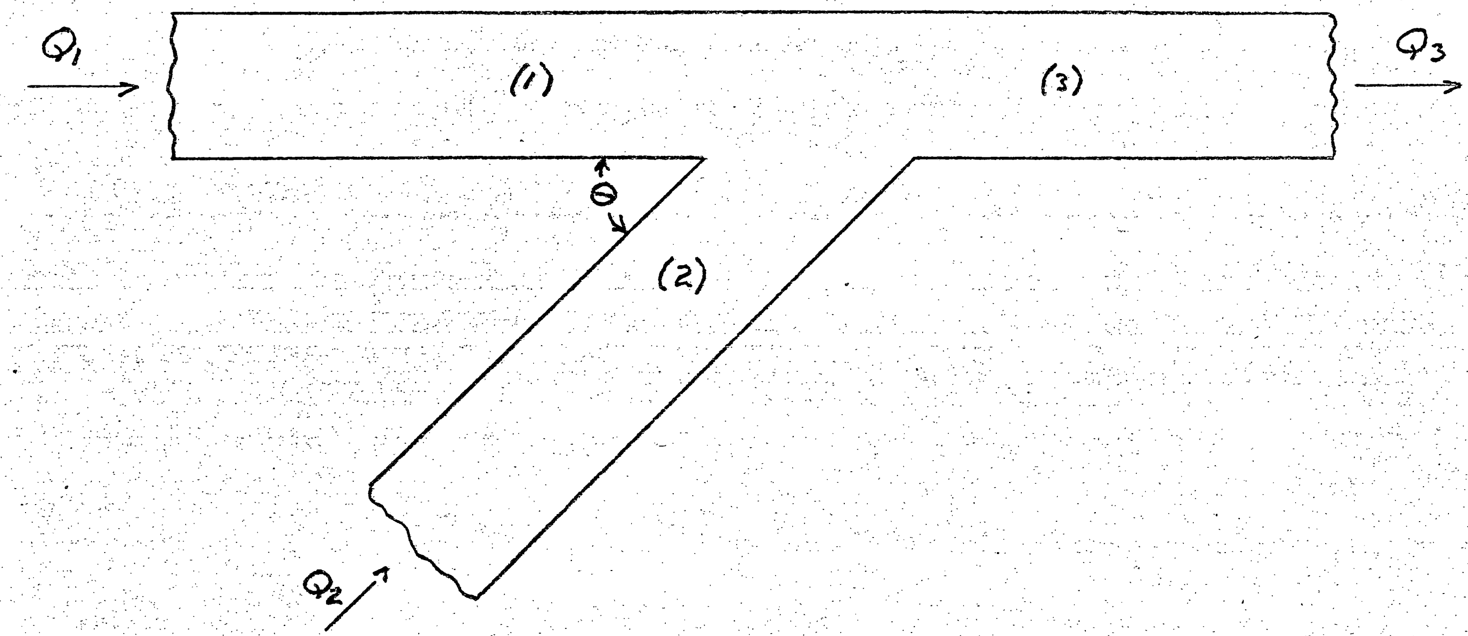


Figure 3: Taylor's rectangular open channel junction (15)

$$\frac{Q_1\gamma}{g}V_1 + \frac{\gamma b(h_1)^2}{2} + \frac{Q_2\gamma}{g}V_2 \cos \theta = \frac{Q_3\gamma}{g}V_3 + \frac{\gamma b(h_3)^2}{2}$$

where:

Q_1, Q_2, Q_3 = flow in respective channels

V_1, V_2, V_3 = velocity in respective channels

h_1, h_3 = depth of flow in respective channels

b = width of channels

θ = angle of entry of channel 2

γ = specific weight of water

g = gravitational constant.

Entrance angles of 45 degrees and 135 degrees were tested experimentally. The theoretical solutions agreed favorably with the experimental results for entrance angles of 45 degrees; however, for angles of 135 degrees, there was rather poor correlation. This lack of agreement for angles of 135 degrees was thought to be the result of the distortion of the velocity distribution below the junction.

Shindala (12) developed an adaption of Sangster's junction box analysis to the analysis of manhole junctions. This adaption involved the switch from a pressurized system to one involving open channel flow. Three situations were investigated:

1. Straight-through junctions with no lateral pipes.
2. Straight-through junctions with one 90 degree lateral and the major flow from the straight-through pipe.
3. Straight-through junctions with one 90 degree lateral

and the major flow from the lateral.

The analysis of the first and second situations was a direct application of the momentum principle with the assumption that the 90 degree lateral contributed no momentum to the downstream pipe. The analysis of the third situation made use of Sangster's experimental data in order to evaluate the momentum contribution of the lateral flow. Tables and curves were prepared to facilitate the solutions for the three cases; however, an error was present in the prepared table of hydraulic elements. The values given as the depth to the centroid of the cross sectional areas in a pipe flowing partly full were actually the distances from the invert of the pipe to the centroid rather than the distance from the water surface to the centroid. This error resulted in incorrect solutions to the example problems used by Shindala to show the applicability of his method.

Shindala did state, however, that because the method was an adaption of solutions for full flow condition, experimental verification should be obtained for the partial flow conditions present in sanitary sewers.

The review of the literature indicates a need for a model study of the characteristics of the hydraulics of a sanitary type manhole junction. There is general agreement that the approach for such an analysis should be based on the conservation of momentum theory. The important factors which should be considered in an analysis of this type would be:

1. Open channel flow is present throughout the transition.

2. The flow is confined in U-shaped channels. The impact and mixing of converging flows must take place within these channels.
3. The lateral channels are curved in the direction of the downstream outlet so that the lateral flow intersects the main flow in some acute angle.
4. The invert elevations of the inlet and outlet pipes may be at different elevations, with a resulting slope in the bottoms of the transition channels.

The works of Shindala and Taylor come closest to considering all of the factors listed above. However, neither work considers all four of these factors. Shindala's work neglects the effects of the lateral flow entering at an angle other than 90 degrees. Taylor's work does not consider a sloping channel bottom. Both of these omissions are considered to be important factors and should be included in a complete investigation of a sanitary sewer transition.

Methods and Materials

Test Equipment

The model investigation was performed in the hydraulics laboratory located in the basement of Norris Hall on the campus of the Virginia Polytechnic Institute. A plan view of the equipment arrangement is presented in Figure 4, page19. The equipment used consisted of the following:

1. One weir box which received the total incoming flow and divided it into a flow to pipe 1 and a flow to pipe 2. The box was 3.0 feet long, 3.0 feet wide, and 3.0 feet high; and was constructed with 3/8-inch varnished exterior grade plywood. A hook gauge which read to one one-thousandth of a foot was mounted on the side of the box in order to measure the water surface elevation. Two weirs were placed in the sides of the box to allow water to flow to pipes 1 and 2.
2. Weir 1, which was located in the side of the weir box, consisted of a 6.5 inch rectangular slot cut in the plywood. The bottom of the weir could be raised or lowered to vary the flow. The flow from this weir fell directly into head box 1.
3. Weir 2, which consisted of a 3.0 inch rectangular weir formed by a slot cut in a 14 gauge galvanized steel plate. This plate was bolted to the side of the weir box so that the flow from the weir fell into a plywood channel which

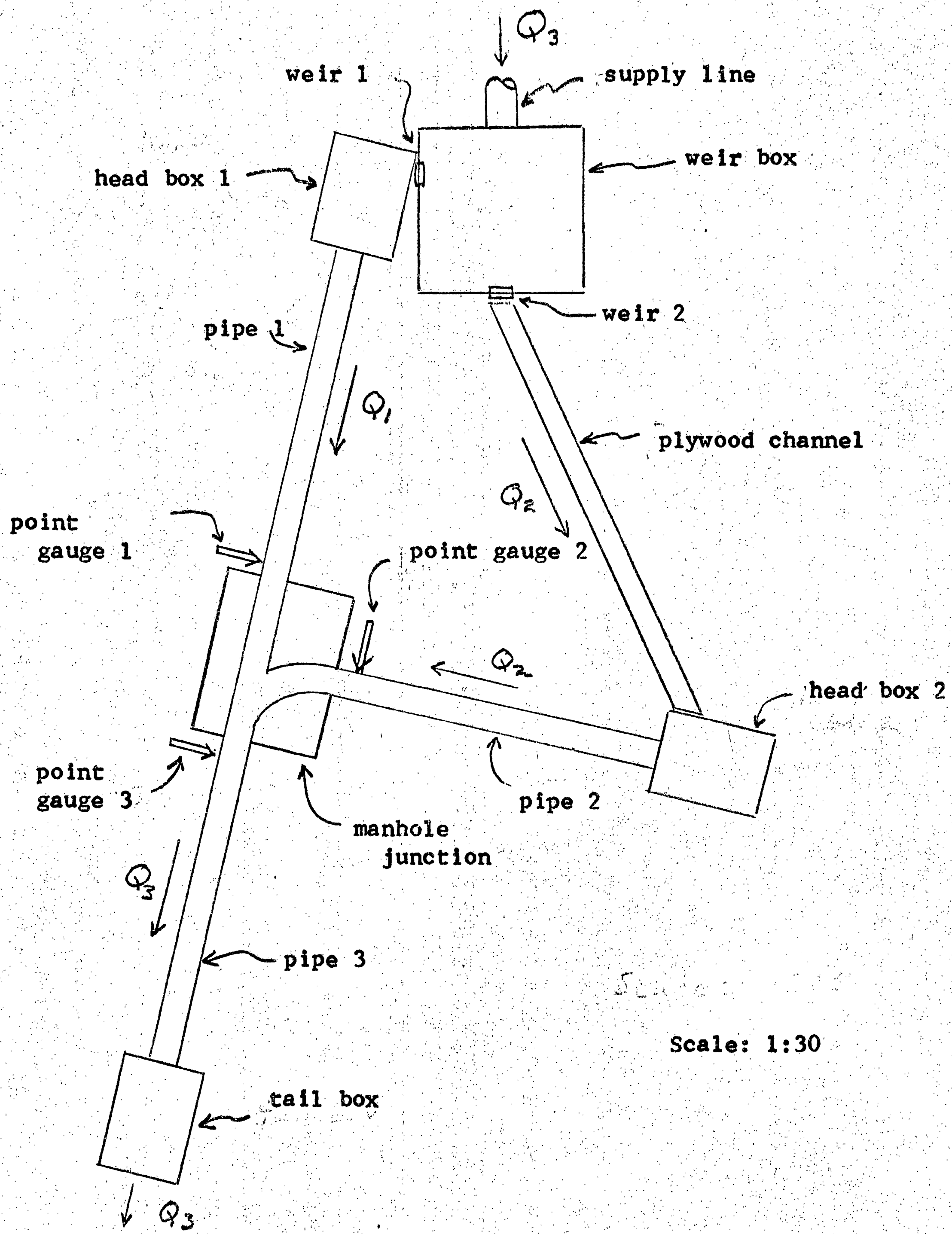


Figure 4: Plan view of experimental test equipment.

carried the water to head box 2.

4. Head boxes 1 and 2, which were located at the upstream ends of pipes 1 and 2 respectively. The boxes received the flows from weirs 1 and 2 and allowed the water to enter the pipes relatively undisturbed. Each box was 18.0 inches wide, 24.0 inches long, and 15.0 inches high; and was constructed with 3/8-inch varnished exterior grade plywood. A hole with a 6.5 inch diameter was cut 8.0 inches above the bottom of the end of each box to allow the pipe to extend part way into the box.

5. Tail box which was located at the downstream end of pipe 3. This box was the same as the two head boxes with the addition of an adjustable weir outlet which allowed the water surface elevation in pipe 3 to be varied.

6. Pipelines 1,2, and 3 each of which was a 6.0 foot long cast acrylic tube with an inside diameter of 6.0 inches and an outside diameter of 6.5 inches, supplied by Gilbert Plastic and Supply Company, Inc. These tubes were placed so that their slopes were zero and their invert elevations were all equal.

7. Point gauges 1,2, and 3 which were mounted above pipes 1, 2, and 3 respectively, such that the points could extend down through a hole drilled in the top of each pipe. These holes were placed 6.0 inches from the ends of the pipes that joined the manhole junction. The points on the

gauges could read to one one-thousandth of a foot.

8. Manhole junction, which conformed as closely as possible to the typical manhole junction. The dimensions were reduced to 3/4 actual size because the 6.0 inch diameter pipes used were smaller than the 8.0 inch minimum size specified by most standards. The materials used were portland cement for the lower portion of the channel and varnished wood for the sides. The geometry of the junction is shown in Figure 5, page 22. The bottom of the channel was horizontal so that all three pipes joined the junction at the same elevation.

All pipe junctions and plywood seams were thoroughly sealed with calking compound to reduce water leakage to a minimum.

Measurements

The total flow to the system was supplied by a constant head tank located on the roof of the building. Water flowed from this tank through a 6 inch line to the weir box. An orifice plate was located in this line, and by measuring the pressure drop across this plate with an manometer and consulting a previously prepared calibration curve, the flow in the line could be determined.

Weir 2 was calibrated by allowing the total flow from the 6 inch supply line to pass over the weir and measuring the head in the weir box with the hook gauge. By this procedure, a list of flows over weir 2 versus hook gauge reading was prepared and a graph relating the two values was plotted. Thus by measuring the head in the weir box and

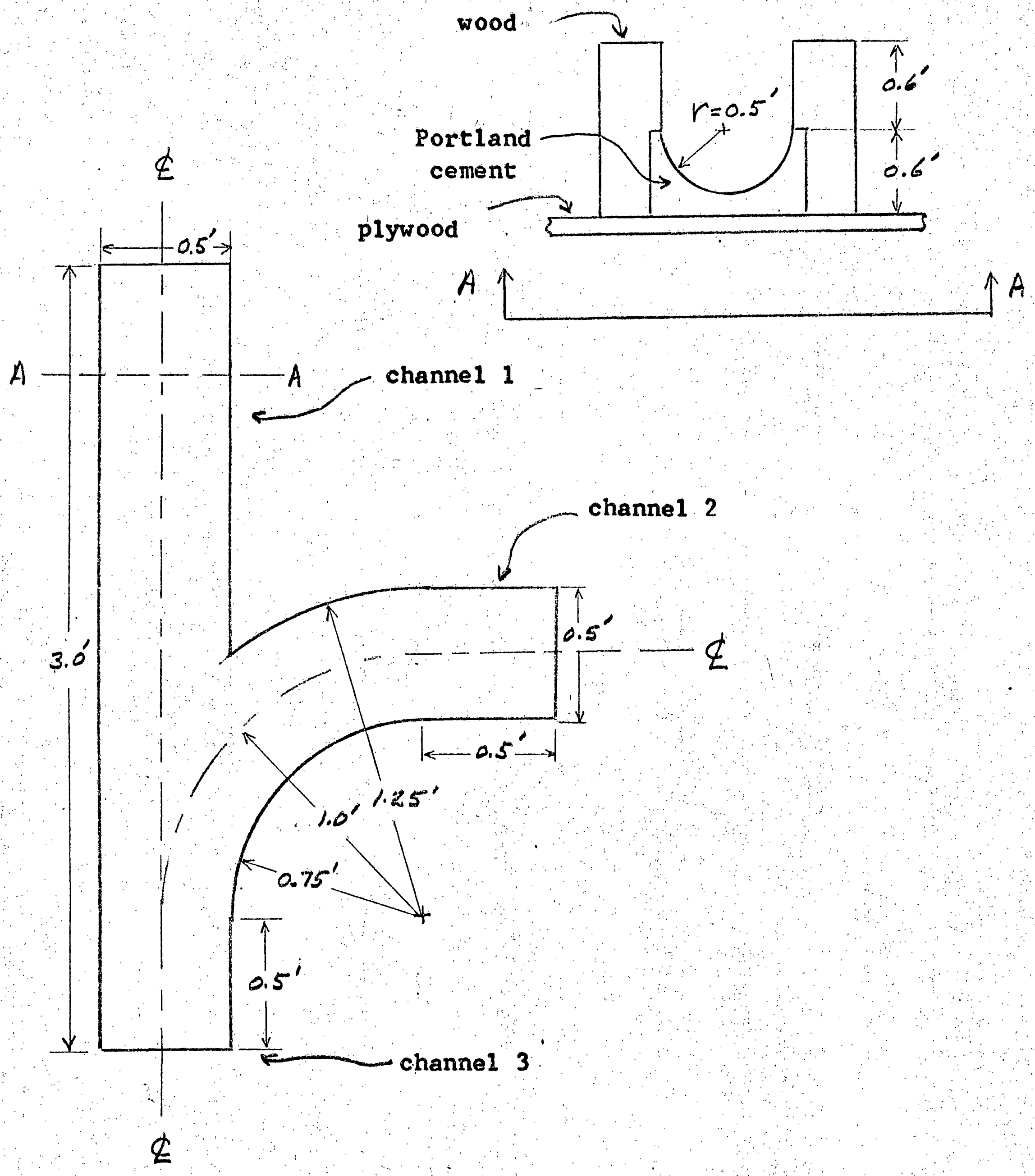


Figure 5: Details of manhole junction

comparing this head with the calibration curve, the flow over weir 2 could be found.

The flow over weir 1 was obtained by subtracting the flow over weir 2 from the flow in the supply line.

The flow depths in each of the three pipes were measured by use of the point gauges. Readings were taken at the invert of each pipe and at the water surface. The difference in these readings was taken as the depth of flow in the pipe.

Methods

The procedure for the collection of data was as follows: Point gauge readings were taken at the inverts of all three pipes with no water flowing. The valve on the supply line was then partially opened to allow water to flow into the weir box. A short time was allowed for the system to reach equilibrium and the orifice plate pressure drop was taken. Weir 1 was then lowered so that the total flow entered pipe 1 and the outlet weir in the tail box was adjusted to give the desired depth of flow in pipe 3. After sufficient time for the flow to become steady, point gauge readings were taken at the water surfaces in pipes 1, 2, and 3. Weir 1 was then raised until a portion of the flow passed over weir 2. The hook gauge reading was taken of the water surface in the weir box. Point gauge readings were retaken of the water surface in the pipes. The procedure of raising weir 1 and taking readings was repeated approximately ten times with the last run passing the total flow over weir 2 and into pipe 2. This series of runs resulted in the following information:

1. the total flow to the system, which remained constant for the series of runs;
2. the depth of flow in pipe 3, which remained constant for the series of runs;
3. the flow in pipes 1 and 2 for each individual run;
4. the depth of flow in pipes 1 and 2 for each individual run.

This entire procedure was repeated six times with four different total flows and downstream depths, and repetitions of two previous runs.

Presentation of Data

The data was collected in the manner described in the chapter "Methods and Materials." The manometer, hook gauge, and point gauge readings taken during the investigation are presented in Appendix 1. The resulting depths and flows found from the experimental measurements are presented in Table 1, page 26. The values found in runs A, B, C, and D were used as a basis for the theoretical investigation and the statistical evaluation of constants. Runs E and F were attempts to duplicate runs C and B respectively. A comparison of the duplications with the original runs showed that the measuring equipment and techniques were sufficiently accurate so that any total run could be reproduced reasonably well. However, it was impractical to try to duplicate any specific observation because of the difficulty in reproducing exactly the same flow ratios between pipes 1 and 2.

Plots of depth of flow in pipes 1, 2, and 3 versus the ratio of the flow in pipe 1 to the flow in pipe 3 are presented in Figures 6 through 9, pages 29 through 32for runs A, B, C, and D. These plots show the relationship between the three flow depths as the flow ratios changed. Four distinctive features are evident in each of the four plots:

1. The depth in channel 3 remained constant for each run.
2. There was always a drop in water surface elevation between the inlets and the outlet regardless of the flow ratio from the inlets.
3. The maximum drop in water surface occurred when the

Table 1: Experimental flows and depths in pipes 1, 2, and 3.

Run no.	Q_3 (cfs)	Q_1 (cfs)	Q_2 (cfs)	d_1 (ft)	d_2 (ft)	d_3 (ft)
A 1	0.190	0.190	0.000	0.280	0.278	0.265
A 2	0.190	0.160	0.030	0.305	0.306	0.265
A 3	0.190	0.135	0.055	0.312	0.311	0.265
A 4	0.190	0.108	0.082	0.314	0.313	0.265
A 5	0.190	0.088	0.102	0.315	0.313	0.265
A 6	0.190	0.067	0.123	0.314	0.310	0.265
A 7	0.190	0.045	0.145	0.313	0.306	0.265
A 8	0.190	0.020	0.170	0.311	0.302	0.265
A 9	0.190	0.000	0.190	0.305	0.295	0.265
B 1	0.136	0.136	0.000	0.236	0.234	0.226
B 2	0.136	0.104	0.032	0.259	0.258	0.226
B 3	0.136	0.086	0.050	0.263	0.260	0.226
B 4	0.136	0.071	0.065	0.263	0.260	0.226
B 5	0.136	0.052	0.084	0.263	0.258	0.226
B 6	0.136	0.040	0.096	0.261	0.256	0.226
B 7	0.136	0.026	0.110	0.260	0.253	0.226
B 8	0.136	0.000	0.136	0.254	0.249	0.226
C 1	0.232	0.232	0.000	0.310	0.304	0.278
C 2	0.232	0.196	0.036	0.338	0.338	0.278
C 3	0.232	0.175	0.057	0.341	0.342	0.278
C 4	0.232	0.152	0.080	0.346	0.344	0.278

Table 1 (continued)

Run no.	Q_3 (cfs)	Q_1 (cfs)	Q_2 (cfs)	d_1 (ft)	d_2 (ft)	d_3 (ft)
C 5	0.232	0.132	0.100	0.349	0.345	0.278
C 6	0.232	0.107	0.125	0.349	0.345	0.278
C 7	0.232	0.090	0.142	0.349	0.344	0.278
C 8	0.232	0.068	0.164	0.347	0.339	0.278
C 9	0.232	0.045	0.187	0.344	0.334	0.278
C 10	0.232	0.027	0.205	0.342	0.329	0.278
C 11	0.232	0.000	0.232	0.335	0.324	0.278
D 1	0.262	0.262	0.000	0.356	0.356	0.347
D 2	0.262	0.230	0.032	0.384	0.383	0.347
D 3	0.262	0.201	0.061	0.388	0.389	0.347
D 4	0.262	0.165	0.097	0.394	0.393	0.347
D 5	0.262	0.137	0.125	0.394	0.392	0.347
D 6	0.262	0.108	0.154	0.394	0.388	0.347
D 7	0.262	0.082	0.180	0.394	0.385	0.347
D 8	0.262	0.055	0.207	0.390	0.382	0.347
D 9	0.262	0.000	0.262	0.383	0.373	0.347
E 1	0.232	0.232	0.000	0.310	0.302	0.280
E 2	0.232	0.206	0.026	0.338	0.337	0.280
E 3	0.232	0.156	0.076	0.345	0.345	0.280
E 4	0.232	0.121	0.111	0.350	0.347	0.280
E 5	0.232	0.097	0.135	0.351	0.345	0.280

Table 1 (continued)

Run no.	Q_2 (cfs)	Q_1 (cfs)	Q_3 (cfs)	d_1 (ft)	d_2 (ft)	d_3 (ft)
E 6	0.232	0.054	0.178	0.348	0.338	0.280
E 7	0.232	0.000	0.232	0.337	0.325	0.280
F 1	0.135	0.135	0.000	0.241	0.235	0.227
F 2	0.135	0.128	0.007	0.260	0.256	0.227
F 3	0.135	0.098	0.037	0.265	0.260	0.227
F 4	0.135	0.057	0.078	0.265	0.259	0.227
F 5	0.135	0.027	0.108	0.262	0.254	0.227
F 6	0.135	0.010	0.125	0.258	0.248	0.227

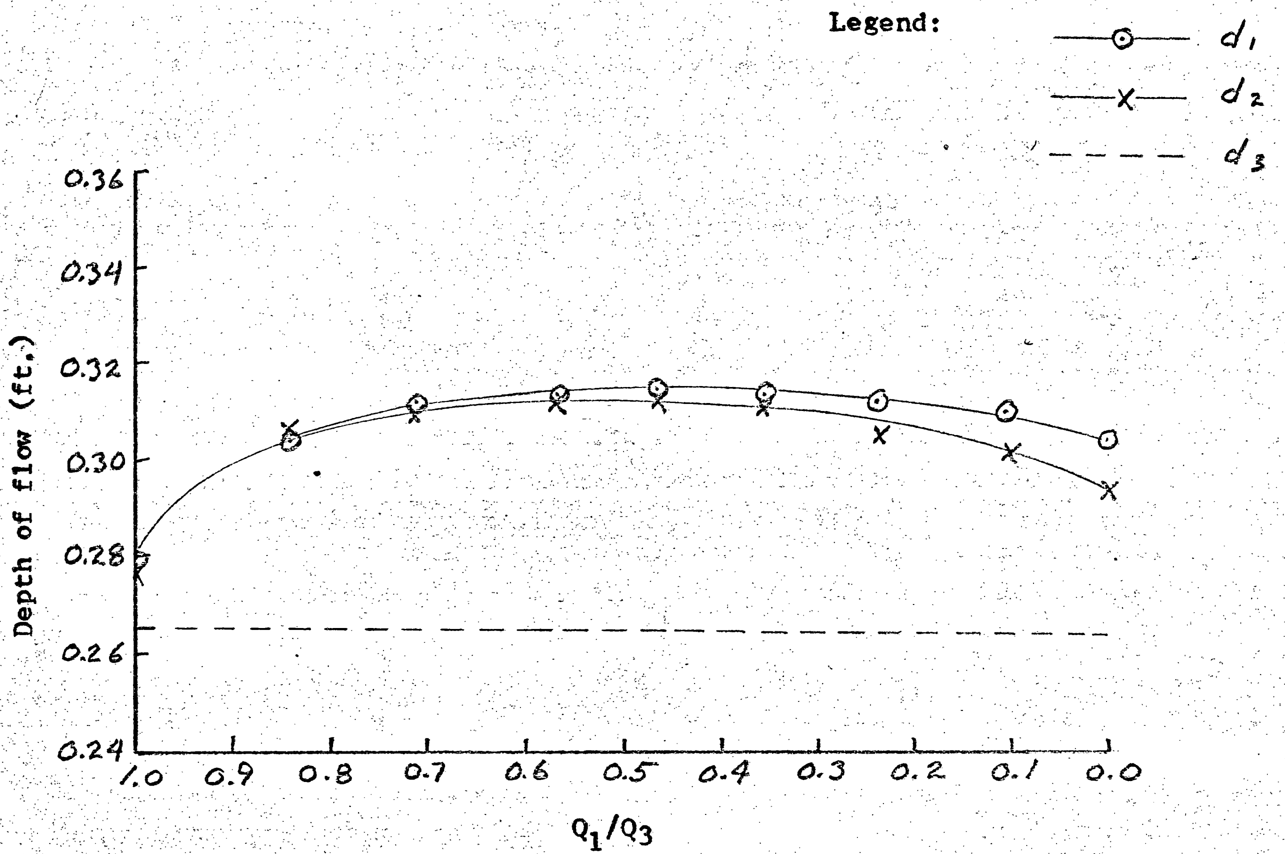


Figure 6: Depth of flow versus Q_1/Q_3 for run A, with Q_3 = 0.190 cfs

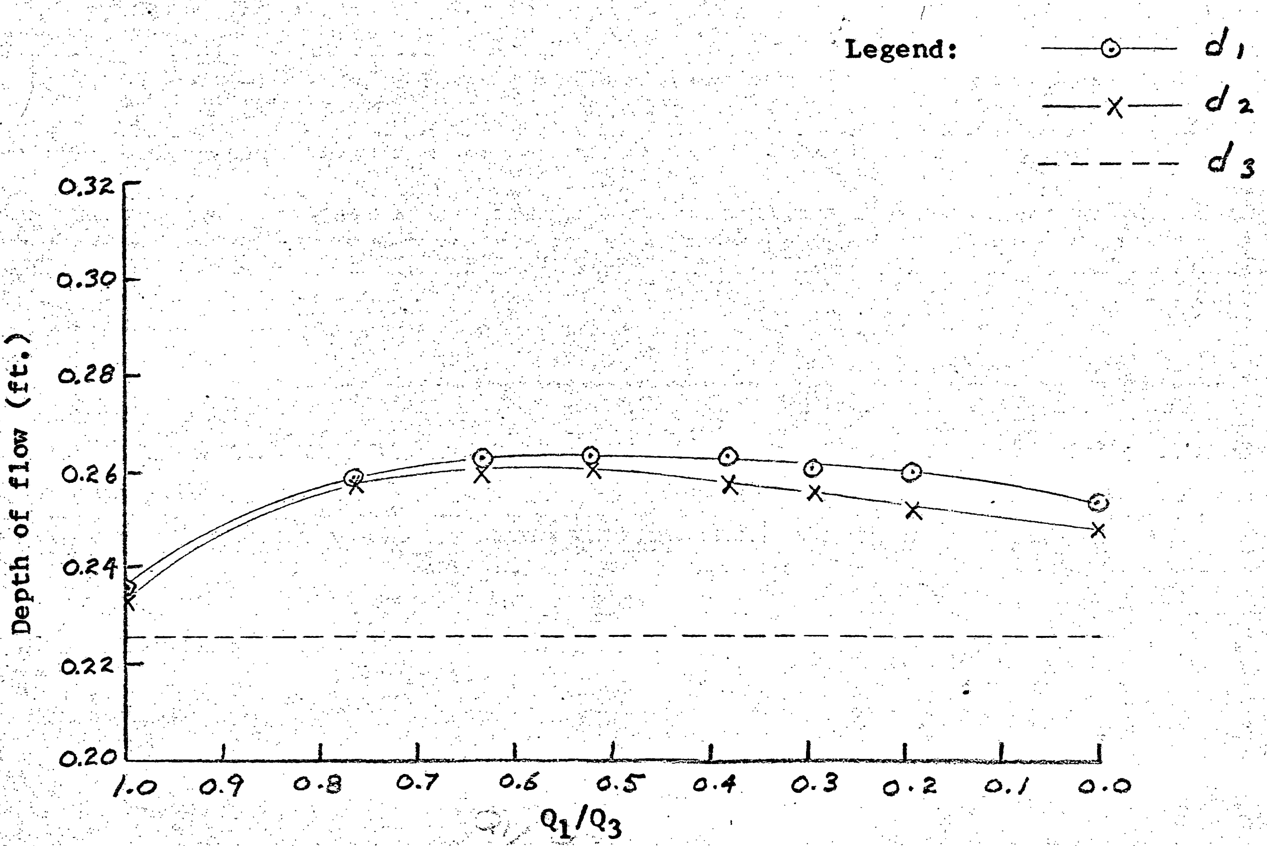


Figure 7: Depth of flow versus Q_1/Q_3 for run B, with Q_3 = 0.136 cfs

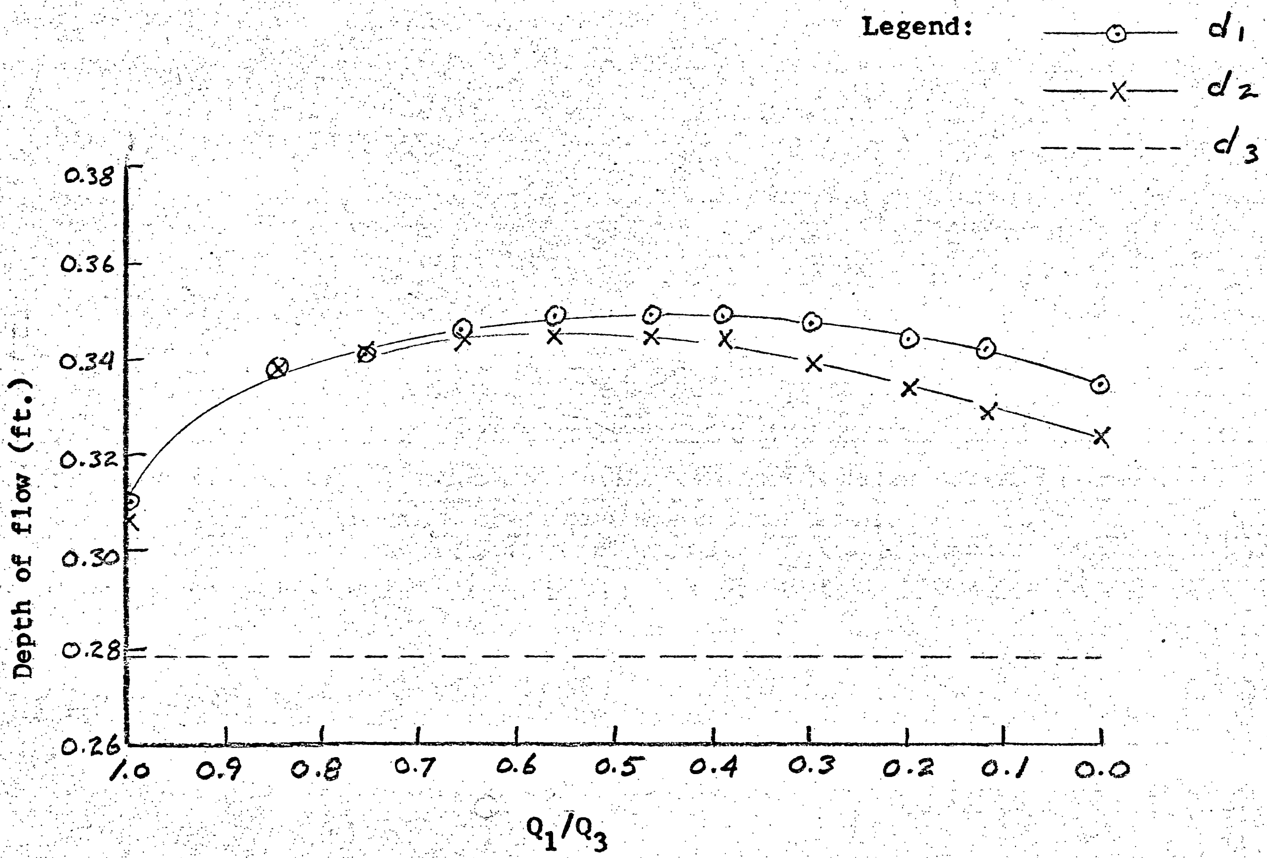


Figure 8: Depth of flow versus Q_1/Q_3 for run C, with Q_3 = 0.232 cfs

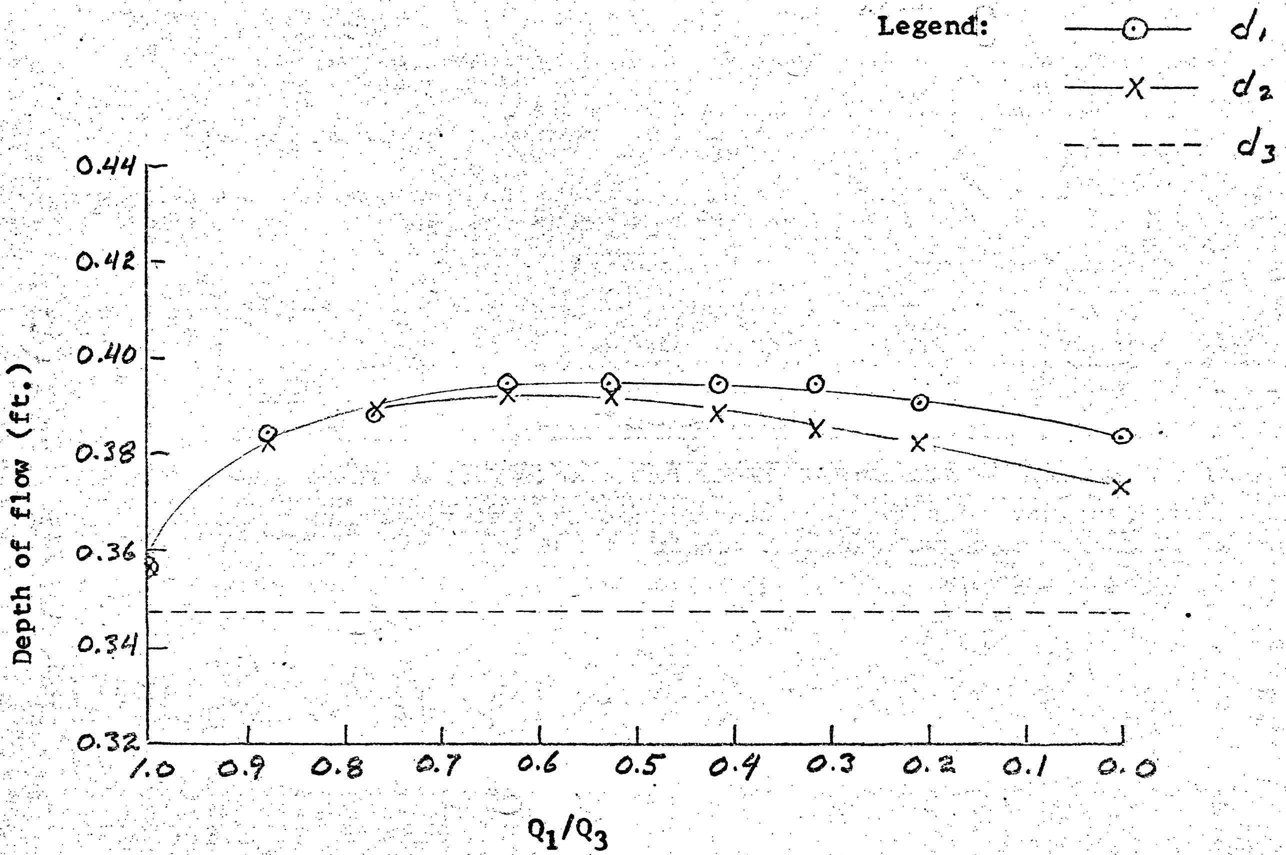


Figure 9: Depth of flow versus Q_1/Q_3 for run D, with Q_3 = 0.262 cfs

quantity of flow from pipes 1 and 2 was approximately equal.

4. The depth in pipe 2 was consistently less than the depth in pipe 1 for any given flow ratio and this difference increased as the fraction of water flowing in pipe 2 increased.

Analysis

Momentum Analysis of Data

The object of this analysis is to develop relationships, by means of the principle of the conservation of momentum, which will describe the experimental values obtained in the model studies.

The principle of the conservation of momentum requires that the change in the linear momentum of a fluid will be equal to the resultant of all the external forces acting on the fluid. The linear momentum of water can be expressed in terms of the momentum flux which is dimensionally the same as a force. If the assumptions are made that the fluid is incompressible and that the momentum flux is constant over the cross-sectional area of flow, the momentum of water passing through a channel section is expressed by:

$$M_i = \frac{\gamma Q_i V_i}{g} \tag{1}$$

where:

Q_i = discharge in channel i (cfs)

V_i = velocity in channel i (ft/sec)

γ = specific weight of water (lbs/ft^3)

g = gravitational constant (ft/sec^2)

The principal external forces acting on the water in the direction of flow can be assumed to be hydrostatic pressure forces and can be given by:

$$P_i = \gamma \bar{y}_i a_i \tag{2}$$

where:

a_i = cross-sectional area of flow in channel i (ft^2)

$\bar{y}_i$ = depth to the centroid of a_i (ft).

The remaining external forces result from external frictional losses between the water and the walls of the channel. Chow (3) assumes the frictional loss to be given by:

$$F_i = h_f' \gamma a_i' \tag{3}$$

where:

h_f' = frictional head

a_i' = average cross-sectional area.

The friction head can be expressed by:

$$h_f' = K_i \frac{(V_i')^2}{2g} \tag{4}$$

where:

K_i = constant for channel i

V_i' = average velocity in channel i.

Combining equations 1, 3, and 4, the expression for the frictional loss becomes:

$$F_i = K_i' M_i \tag{5}$$

where:

K_i' = constant for channel i

M_i = momentum in channel i.

The application of the momentum equation to the manhole junction under study along the center lines of pipes 1 and 3 results in the following equation:

$$M_1 + M_2' - M_3 = P_3 - P_1 - P_2' + F_1 + F_2' + F_3 \tag{6}$$

where:

M_2' = contribution of momentum from channel 2 to the line of action

F_2' and P_2' = contributions of the external forces in channel 2 to the line of action.

If the forces from channel 2 are assumed to be acting on a plane which is parallel to the centerline of channels 1 and 3, the hydrostatic force, P_2, does not contribute any force in the downstream direction. The forces M_2' and F_2' are considered to be some fraction, X_2, of the total M_2 and F_2 in channel 2. By using equation 5 to substitute for the friction forces, equation 6 becomes:

$$C_1 M_1 + C_2 M_2 - C_3 M_3 = P_3 - P_1 \tag{7}$$

where:

$$C_1 = 1 - K_1'$$

$$C_2 = (1 - K_2') X_2$$

$$C_3 = 1 + K_3.$$

Three forms of equation 7 were investigated.

1. The first form resulted from the assumption that all of the frictional losses could be neglected.

$$X_2 M_2 = M_3 - M_1 + P_3 - P_1 \tag{8}$$

2. The second form resulted from the assumption that only the frictional loss in channel 3 should be neglected.

$$C_1 M_1 + C_2 M_2 = M_3 + P_3 - P_1 \tag{9}$$

3. The third form resulted from the assumption that three frictional losses should be included.

$$C_1 M_1 + C_2 M_2 - C_3 M_3 = P_3 - P_1 \tag{10}$$

The unknown constants, X_2, C_1, C_2, and C_3, in each of the three equations were determined by performing a multiple linear regression using the experimental values obtained in the model studies. For each of the 37 experimental data points found, the values of M_1, M_2, M_3, P_1, and P_3 were calculated by using formulas 1 and 2. The regressions were performed on equations 8, 9, and 10 by the use of the BIMED 09 Stepwise Multiple Regression computer program. This program is a modification of a program originally written by M. A. Efraymson of the Esso Research and Engineering Company. The purpose of the program was to perform a multiple linear regression on the input data. An option in the program which allowed the constant term in the regression to be set equal to zero was exercised so that only values of the unknowns were obtained for each equation.

The results of the regression analysis are:

1. Equation 8

$$X_2 = 0.758$$

Resulting equation:

$$M_1 + 0.758\, M_2 - M_3 = P_3 - P_1 \tag{11}$$

2. Equation 9

$$C_1 = 0.939$$

$$C_2 = 0.767$$

Resulting equation:

$$0.939\, M_1 + 0.767\, M_2 - M_3 = P_3 - P_1 \tag{12}$$

3. Equation 10

$C_1 = 0.655$

$C_2 = 0.458$

$C_3 = 0.806$

Resulting equation:

$$0.655\ M_1 + 0.458\ M_2 - 0.806\ M_3 = P_3 - P_1 \qquad (13)$$

In order to determine how well equations 11, 12, and 13 could predict actual flow conditions, these equations were used to predict the upstream flow depths in pipe 1 using the quantities of flow in each of the three pipes and the depth of flow in pipe 3. A further simplifying assumption was made that the depths of flow in pipes 1 and 2 were equal for any given set of flow conditions. It was felt that this simplifying assumption was justified because the backwater depth conditions would be approximately the same for both channels under subcritical flows.

The results of the flow predictions using equations 11, 12, and 13 are presented in table 2, page 39.

Determination of the most satisfactory form of the three equations investigated was based on two considerations. The first was whether the magnitude and sign of the constants found by the regression determination conformed with what would be expected from a hydraulic standpoint. The second consideration was how well the final equation predicted the upstream flow depths.

Equation 11 was the result of the assumption that all friction losses could be neglected. The term X_2 which represented the fraction of M_2 acting in the downstream direction was found to be 0.758. This value was considered to be reasonable considering the geometry of the junction.

Table 2: Results of flow depth predictions using equations 11, 12, and 13.

Run no.	actual d_1 (ft)	Equation 11		Equation 12		Equation 13	
		calculated d_1 (ft)	difference (ft)	calculated d_1 (ft)	difference (ft)	calculated d_1 (ft)	difference (ft)
A 1	0.280	0.266	0.014	0.276	0.004	0.285	0.005
A 2	0.305	0.300	0.005	0.304	0.001	0.303	0.002
A 3	0.312	0.314	0.002	0.316	0.004	0.311	0.001
A 4	0.314	0.322	0.008	0.323	0.009	0.316	0.002
A 5	0.315	0.324	0.009	0.325	0.010	0.318	0.003
A 6	0.314	0.323	0.009	0.323	0.009	0.318	0.004
A 7	0.313	0.319	0.006	0.319	0.006	0.316	0.003
A 8	0.311	0.310	0.001	0.309	0.002	0.311	0.000
A 9	0.305	0.297	0.008	0.297	0.008	0.304	0.001
B 1	0.236	0.227	0.009	0.234	0.002	0.242	0.006
B 2	0.259	0.260	0.001	0.262	0.003	0.259	0.000
B 3	0.263	0.268	0.005	0.269	0.006	0.264	0.001
B 4	0.263	0.271	0.008	0.272	0.009	0.266	0.003
B 5	0.263	0.271	0.008	0.272	0.009	0.267	0.004
B 6	0.261	0.270	0.009	0.270	0.009	0.264	0.004

Table 2 (continued)

Run no.	actual d_1 (ft)	Equation 11 calculated d_1 (ft)	Equation 11 difference (ft)	Equation 12 calculated d_1 (ft)	Equation 12 difference	Equation 13 calculated	Equation 13 difference
B 7	0.260	0.266	0.006	0.266	0.006	0.264	0.004
B 8	0.254	0.251	0.003	0.250	0.004	0.256	0.002
C 1	0.310	0.279	0.031	0.294	0.016	0.306	0.004
C 2	0.338	0.326	0.012	0.331	0.007	0.328	0.010
C 3	0.341	0.339	0.002	0.342	0.001	0.336	0.005
C 4	0.346	0.348	0.002	0.350	0.004	0.342	0.004
C 5	0.349	0.353	0.004	0.354	0.005	0.345	0.004
C 6	0.349	0.355	0.006	0.356	0.007	0.347	0.002
C 7	0.349	0.355	0.006	0.356	0.007	0.348	0.001
C 8	0.347	0.353	0.006	0.353	0.006	0.346	0.001
C 9	0.344	0.347	0.003	0.347	0.003	0.343	0.001
C 10	0.342	0.340	0.002	0.339	0.003	0.339	0.003
C 11	0.335	0.323	0.012	0.322	0.013	0.330	0.005
D 1	0.356	0.348	0.008	0.356	0.000	0.366	0.010

Table 2 (continued)

Run no.	actual d_1 (ft)	Equation 11		Equation 12		Equation 13	
		calculated d_1 (ft)	difference (ft)	calculated d_1 (ft)	difference (ft)	calculated d_1 (ft)	difference (ft)
D 2	0.384	0.374	0.010	0.379	0.005	0.380	0.004
D 3	0.388	0.389	0.001	0.393	0.005	0.389	0.001
D 4	0.394	0.401	0.007	0.402	0.008	0.397	0.003
D 5	0.394	0.405	0.011	0.406	0.012	0.400	0.006
D 6	0.394	0.406	0.012	0.407	0.013	0.401	0.007
D 7	0.394	0.404	0.010	0.404	0.010	0.400	0.006
D 8	0.390	0.399	0.009	0.399	0.009	0.397	0.007
D 9	0.383	0.377	0.006	0.376	0.007	0.385	0.002

The use of equation 11 to predict the upstream depths of flow gave rather poor results when the total flow entered the junction from pipe 1. The poor results were thought to be due to the failure to consider the friction losses in channel 1.

Equation 12 was the result of the attempt to include friction loss terms for both channels 1 and 2. The term C_1 reflected the summation of the momentum force and the frictional force loss in channel 1 and was found to be 0.939. The term C_2 reflected the summation of the momentum force and the frictional loss force contributed by the flow in channel 2 and was found to be 0.767. These values were considered to be reasonable for the junction under study. The use of equation 12 to predict the upstream depths gave better results than equation 11, especially when the total flow entered the junction from channel 1.

Equation 13 was the result of the attempt to improve on the results of equation 12 by including a friction loss term for channel 3 as well as for channels 1 and 2. The term C_3 reflected the summation of the momentum forces and frictional forces in channel 3 and was found to be 0.806. This value was not considered to be a reasonable value because according to the mathematical development of equation 7, the term C_3 should be greater than 1.0. For this reason, equation 13 was considered to be unacceptable.

In order to determine the difference between the ability of equations 11, 12, and 13 to estimate the upstream depths, the standard error of estimate,

$$S = \sqrt{\frac{\sum (d_1 - d_{1\,est.})^2}{N}}$$

was calculated for each equation. The results of these calculations are:

Equation 11: $S = 0.0090$ feet

Equation 12: $S = 0.0074$ feet

Equation 13: $S = 0.0043$ feet.

The standard error of estimate is an indication of how large the estimating error is, and for this reason equation 12 was considered to be better for estimating the observed data than equation 11. Therefore, equation 12 was chosen as the best equation of the three equations tested for use in manhole design.

<u>Analysis of Invert Drop</u>

All model studies were conducted with horizontal channels; however, for practical purposes of design, an expression is desired which relates the effect of dropping the invert of pipe 3 on the external forces in the momentum equation.

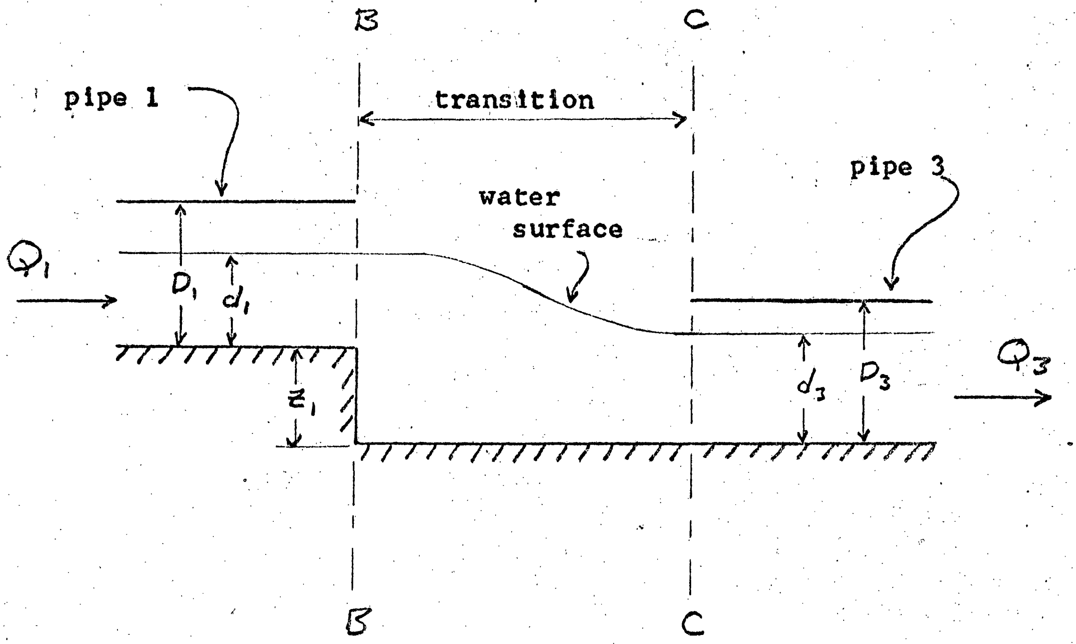
For purposes of simplicity, a straight-through manhole with no lateral junction was investigated first. The flow and channel characteristics are shown in Figure 10, page 44. The momentum equation was applied between sections B-B and C-C. The momentum force at B-B is the same as the momentum force in pipe 1. The cross-sectional area at section B-B can be divided into three areas, a_{11}, a_{12}, and a_{13}, as shown in Figure 10, page 44. The hydrostatic force on this area is given by:

$$P_{B-B} = \gamma \left[a_{11} (\bar{y}_{11}) + a_{12} (\bar{y}_{12}) + a_{13} (\bar{y}_{13}) \right] \qquad (14)$$

where:

P_{B-B} = the hydrostatic force on section B-B

$\bar{y}_{11}, \bar{y}_{12}, \bar{y}_{13}$ = the depth to the centroid of areas a_{11}, a_{12}, and a_{13} respectively.



Area of flow at B - B

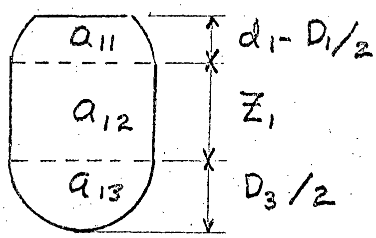


Figure 10: **Channel characteristics with drop in pipe 3.**

By assuming D_1 is equal to D_3, equation 14 becomes:

$$P_{B-B} = P_1 + \gamma \left\{ \left[D_1 d_1 + D_1^2 \left(\frac{\pi - 2}{4}\right) \right] Z_1 + \frac{D1}{2} Z_1^2 \right\} \qquad (15)$$

where:

P_1 = the hydrostatic force in pipe 1 as given by equation 2

D_1 = the diameter of pipe 1 (ft.)

Z_1 = the difference in invert elevation between pipes 1 and 2 (ft.).

Since the hydrostatic force contributed by a lateral pipe is considered to be zero, no additional force would be contributed from a lateral by a change in invert elevation. Therefore, the additional force, P_D, which would result from a change in invert elevation, can be obtained by subtracting the hydrostatic force in pipe 1, P_1, from the total hydrostatic force with the drop, P_{B-B}. The resulting equation is:

$$P_D = \gamma \left[(D_1 d_1 + 0.285\, D_1^2)\, Z_1 + (0.5\, D_1)\, Z_1^2 \right] \qquad (16)$$

If the assumption is made that the constants C_1 and C_2 remain the same for a junction with unequal invert elevations, the additional force required, P_D, to pass a given upstream flow condition through a manhole can be found from a modification of equation 7.

$$P_D = M_3 + P_3 - (0.939\, M_1 + 0.767\, M_2 + P_1) \qquad (17)$$

The required invert drop, Z_1, which is needed to pass a specific flow condition through a manhole can be found by the simultaneous solution of equations 16 and 17. It should be remembered that only positive values of Z_1 would be used for design purposes.

In designing a manhole junction, the design would usually be based on the worst condition that could occur in that junction. The plots of depth of flow versus the flow ratio between pipes 1 and 3 indicate that the largest drop in water surface through a junction for any given flow occurs when the flow from pipes 1 and 2 are approximately equal. Therefore the design of a manhole junction should be based on a condition of equal upstream flows.

To illustrate the use of equations 16 and 17, a tabular solution has been prepared, Figure 11, page 47, for a system of three eight-inch pipes. The assumptions used in developing this solution are:

1. All three pipes are 8.0 inches in diameter.
2. Pipes 1 and 2 are at the same elevation.
3. The flows from pipes 1 and 2 are equal.

To illustrate the procedure for using the tabular solution, an example problem will be worked.

Problem: Find the required drop in invert elevation for a manhole of three 8-inch pipes so that the flow depths in all three pipes do not exceed 0.8 of the diameter with a maximum flow of 0.6 cubic feet per second.

Solution: $d_1/D = 0.8$

$d_3/D = 0.8$

$Q_3 = 0.6$ cfs

From Figure 11, page 47.

$Z_1 = 0.048$ feet

Q_3 (cfs)	d_1/D_1	d_3/D_3			
		0.4	0.6	0.8	1.0
0.4	0.2	0.074	0.146	0.369	0.628
	0.4	0.103	0.137	0.252	0.398
	0.6	0.016	0.041	0.123	0.232
	0.8	0.000	0.000	0.022	0.109
	1.0	0.000	0.000	0.000	0.014
0.5	0.2		0.041	0.261	0.534
	0.4		0.153	0.253	0.394
	0.6		0.063	0.134	0.238
	0.8		0.000	0.033	0.118
	1.0		0.000	0.000	0.022
0.6	0.2		0.000	0.111	0.410
	0.4		0.171	0.253	0.389
	0.6		0.089	0.147	0.246
	0.8		0.001	0.048	0.127
	1.0		0.000	0.000	0.032
0.7	0.4		0.193	0.254	0.384
	0.6		0.119	0.162	0.255
	0.8		0.030	0.064	0.139
	1.0		0.000	0.000	0.043

Figure 11: Tabular solution for the required drop in invert elevation, Z_1 for a system of 8-inch pipes.

Discussion

The object of the chapter "Analysis" was to develop a mathematical relationship which would satisfy the data obtained in the model study of a manhole junction. A review of the literature indicated that an approach using the principle of the conservation of momentum was the most practical because of the difficulty in applying the energy equation to a junction of the type under study. The momentum principle was used in the analysis; however, the application of the equation was complicated by the curvature in the side channel and the lack of understanding of the nature of the friction and impact forces.

Three forms of the momentum equation were investigated. One form of the equation, based on the assumption of no frictional losses in the system, gave poor estimates of the flow depths. A second equation which included factors for frictional forces in each of the three channels based on a linear relationship of the momentum force in each channel was rejected because the statistically determined value of the friction force in channel three was found to be negative. A compromise equation, equation 12, which included factors for frictional losses in the two incoming channels was chosen as being the best of the three equations tested.

The negative value of the frictional loss found for channel 3 indicated that the frictional losses in the channels could possibly be described by something other than a linear function of the momentum in the channels.

It was observed in the chapter "Presentation of Data," that the depth of flow in pipe 2 was consistently less than the depth in pipe 1. The difference in depth was probably because of the momentum force in pipe 2 which was not accounted for in the downstream direction. An analysis of the difference in depths was not considered justified because the difference was small in comparison to the total depths involved.

In order to illustrate the relationship between equation 12 and the experimental data, Table 2, page 39, presented a list of calculated upstream depths versus actual experimental depths for the 37 experimental observations used. It can be noted from this list that the average difference between the actual and calculated values is 0.0065 feet and the standard error of estimate,

$$S = \sqrt{\frac{\sum(d_1 - d_1 \text{ est.})^2}{N}}$$

was found to be 0.0074 feet.

In a typical manhole design, the invert elevation of the outlet pipe is lower than the invert elevation of the inlet pipes. In order to relate this drop in invert to the momentum principle, equation 16 was developed, which predicted the additional force contributed in the downstream direction resulting from a drop in invert elevation. In order to design a manhole junction, equations 16 and 17 would be solved simultaneously for the required invert drop. The solution to these equations involves the evaluation of many factors and the solution of a quadratic equation. For this reason, a tabular solution was presented for a system of eight-inch pipes. It should be remembered that no experimental verification of equation 16 was attempted, and that the tabular solution was presented only to illustrate how the relationships could be used if verification of equation 16 was obtained. Furthermore, the evaluation of the constants C_1 and C_2 in equation 17 made use of the data from only one junction configuration and it is not known how C_1 and C_2 would be affected by a drop in the invert elevation of pipe 3.

Conclusions

This investigation has led to the following conclusions:

1. An open channel manhole junction with subcritical flow is subject to analysis by the momentum principle.
2. The contribution of momentum from a lateral pipe should be included in an analysis of manhole junctions.
3. Further investigation is needed on the effects of:
   a. Different pipe sizes.
   b. Different invert elevations.
   c. Different channel geometries.
   d. Larger total flows and velocities.

Summary

Manholes are the most common appurtenance in sewer systems and they are invariably sites of hydraulic transitions. The present practice in manhole design is to apply empirical allowances for loss of head incurred in the manhole with little regard for the quantities of flow or the velocities involved.

The object of this investigation was the development of hydraulic relationships by the use of an experimental model study of a flow-through manhole with one lateral connection. An attempt was made to use the conservation of momentum principle to develop a mathematical relationship which would satisfy the data obtained in the model study.

Three forms of the momentum equation were developed by statistical evaluation of loss coefficients. Equation 13 was chosen for its ability to predict flow depths and its consistency with hydraulic concepts.

$$0.655\ M_1 + 0.458\ M_2 - 0.806\ M_3 = P_3 - P_1 \qquad (13)$$

where:

M_i = momentum in channel i

P_i = hydrostatic pressure in channel i (see Figure 5, page 22)

In order to describe the effect of dropping the invert elevation of the outlet pipe on the momentum equation, equation 16 was developed.

$$P_D = \gamma \left[(D_1\ d_1 + 0.285\ D_1^2)\ Z_1 + (0.5\ D_1)\ Z_1^2 \right] \qquad (16)$$

where:

P_D = force contributed by a rise of invert elevation of pipe 1 by Z_1 feet.

D_1 = diameter of pipe 1 (ft)

d_1 = depth of flow of water in pipe 1 (ft).

Equation 16 relates the additional force contributed to the system by a rise in the invert elevation of pipe 1. The use of equation 16 together with equation 13 to find the required invert elevation difference between the inlet and outlet pipes in a manhole was then illustrated in an example problem.

It was concluded from the model study and the development of equations 13 and 16 that further investigation was needed before these equations could be recommended for design purposes.

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Appendix 1

Experimental observations for runs A through F.

Run no.	Monometer reading	Hook gauge reading	Point gauge reading 1	2	3
A 0	0.00		1.547	1.422	2.520
A 1	2.00	0.000	1.827	1.728	2.784
A 2	2.00	1.298	1.852	1.728	2.784
A 3	2.00	1.328	1.859	1.733	2.784
A 4	2.00	1.361	1.861	1.735	2.784
A 5	2.00	1.390	1.862	1.735	2.784
A 6	2.00	1.419	1.861	1.732	2.784
A 7	2.00	1.452	1.860	1.728	2.784
A 8	2.00	1.481	1.858	1.724	2.784
A 9	2.00	———	1.852	1.717	2.784
B 0	0.00		1.546	1.422	2.520
B 1	1.00	0.000	1.782	1.656	2.746
B 2	1.00	1.300	1.805	1.680	2.746
B 3	1.00	1.320	1.809	1.682	2.746
B 4	1.00	1.339	1.809	1.682	2.746
B 5	1.00	1.364	1.809	1.680	2.746
B 6	1.00	1.382	1.807	1.678	2.746
B 7	1.00	1.401	1.806	1.675	2.746
B 8	1.00	———	1.800	1.671	2.746

Appendix 1 (continued)

Run no.	Monometer reading	Hook gauge reading	Point gauge reading		
			1	2	3
C 0	0.00		1.546	1.423	2.520
C 1	3.00	0.000	1.856	1.727	2.798
C 2	3.00	1.303	1.884	1.761	2.798
C 3	3.00	1.330	1.887	1.765	2.798
C 4	3.00	1.358	1.892	1.767	2.798
C 5	3.00	1.389	1.895	1.768	2.798
C 6	3.00	1.422	1.895	1.768	2.798
C 7	3.00	1.450	1.895	1.767	2.798
C 8	3.00	1.481	1.893	1.762	2.798
C 9	3.00	1.512	1.890	1.757	2.798
C 10	3.00	1.536	1.888	1.752	2.798
C 11	3.00	———	1.881	1.747	2.798
D 0	0.00		1.544	1.425	2.517
D 1	3.80	0.000	1.900	1.781	2.864
D 2	3.80	1.300	1.928	1.808	2.864
D 3	3.80	1.335	1.932	1.814	2.864
D 4	3.80	1.377	1.938	1.818	2.864
D 5	3.80	1.421	1.938	1.817	2.864
D 6	3.80	1.466	1.938	1.813	2.864
D 7	3.80	1.505	1.938	1.810	2.864
D 8	3.80	1.540	1.934	1.807	2.864
D 9	3.80	———	1.927	1.798	2.864

Appendix 1 (continued)

Run no.	Monometer reading	Hook gauge reading	Point gauge reading 1	2	3
E 0	0.00		1.544	1.425	2.517
E 1	3.00	0.000	1.854	1.727	2.797
E 2	3.00	1.294	1.882	1.762	2.797
E 3	3.00	1.354	1.889	1.770	2.797
E 4	3.00	1.401	1.894	1.772	2.797
E 5	3.00	1.440	1.895	1.770	2.797
E 6	3.00	1.501	1.892	1.763	2.797
E 7	3.00	——	1.881	1.750	2.797
F 0	0.00		1.549	1.419	2.527
F 1	1.00	0.000	1.790	1.654	2.754
F 2	1.00	1.250	1.809	1.675	2.754
F 3	1.00	1.306	1.814	1.679	2.754
F 4	1.00	1.356	1.814	1.678	2.754
F 5	1.00	1.398	1.811	1.673	2.754
F 6	1.00	1.422	1.807	1.667	2.754

Appendix 2

Hydraulic elements for partially filled circular pipes.

d/D	a/A	$\bar{y}$/D
0.00	0.0000	0.0000
0.02	0.0048	0.0144
0.04	0.0134	0.0177
0.06	0.0245	0.0251
0.08	0.0375	0.0328
0.10	0.0520	0.0401
0.12	0.0680	0.0483
0.14	0.0851	0.0570
0.16	0.1033	0.0650
0.18	0.1224	0.0734
0.20	0.1424	0.0816
0.22	0.1631	0.0900
0.24	0.1846	0.0983
0.26	0.2066	0.1076
0.28	0.2292	0.1152
0.30	0.2523	0.1237
0.32	0.2759	0.1323
0.34	0.2998	0.1408
0.36	0.3241	0.1496
0.38	0.3487	0.1583
0.40	0.3735	0.1672

d/D	a/A	$\bar{y}$/D
0.42	0.3986	0.1761
0.44	0.4238	0.1850
0.46	0.4491	0.1940
0.48	0.4745	0.2031
0.50	0.5000	0.2120
0.52	0.5255	0.2215
0.54	0.5509	0.2308
0.56	0.5763	0.2401
0.58	0.6015	0.2496
0.60	0.6264	0.2594
0.62	0.6513	0.2691
0.64	0.6758	0.2788
0.66	0.7002	0.2888
0.68	0.7241	0.2990
0.70	0.7476	0.3092
0.72	0.7708	0.3196
0.74	0.7934	0.3333
0.76	0.8155	0.3411
0.78	0.8369	0.3520
0.80	0.8576	0.3634
0.82	0.8776	0.3748

Appendix 2 (continued)

d/D	a/A	$\overline{y}/D$
0.84	0.8967	0.3866
0.86	0.9149	0.3987
0.88	0.9320	0.4113
0.90	0.9479	0.4241
0.92	0.9626	0.4376
0.94	0.9756	0.4516
0.96	0.9866	0.4665
0.98	0.9953	0.4824
1.00	1.0000	0.5000

HYDRAULIC MODEL STUDY OF A MANHOLE JUNCTION

by

David G. Parker

ABSTRACT

The object of this investigation was the experimental investigation of the hydraulics of a flow-through manhole with one lateral connection. An attempt was made to use the conservation of momentum principle to develop a mathematical relationship which satisfied the data obtained in the model study. Three forms of the momentum equation were developed by statistical evaluation of loss coefficients. The best of the three equations was chosen on the basis of its ability to predict flow depths and its consistancy with hydraulic concepts. A procedure was then developed for using this equation to obtain a value of the required outlet pipe invert drop for a given set of flow conditions. The use of the procedure was illustrated by the solution of an example problem of a system of three pipes.