PROBABILISTIC JUNCTION ANALYSIS IN STORM DRAIN HYDRAULICS

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A junction structure analysis computer program is developed for use in civil engineering designs of storm drain pipe systems. This program is novel because it enables the engineer to evaluate combinations of quasi steady-state flow regimes within the junction structure. The program utilizes an analytic solution of the pressure-plus-momentum balance equation for computing energy loss values. Because storm drain systems are designed to accommodate maximum test flow conditions, use of the proposed approach will enable a more consistent design than given by the usual single analysis methods in common use.

INTRODUCTION

A common hydraulic problem is the computation of energy losses at the confluence of lateral storm pipe drains to the collector drain (i.e., a junction structure). The usual procedure in the analysis is to assume steady flow throughout the entire storm drain system, and then compute cumulative energy losses. One such energy loss is the junction structure energy loss, h_j, and is usually computed based upon a balance of pressure-plus-momentum in order to estimate the change in flow characteristics, and then other losses such as friction are included (for example see the Los Angeles County Design Manual, 1978, which contains procedures in wide use in the United States). Because all flows are assumed to be constant, the estimation of h_j is a straight-forward procedure.

The focus of this note is to examine the steady-flow assumption used in the usual estimation of h_j, and then develop a procedure to include variations in the combination of inflows to the junction structure so that the sensitivity of h_j to assumed inflow combinations may be determined.

MATHEMATICAL MODEL DEVELOPMENT

The usual procedure in computing h_j is to assume a prescribed set of inflows into the junction structure, and then compute a pressure-plus-momentum balance through the structure. Referencing Fig. 1, the mainline flows are the upstream and downstream flows, Q_1 and Q_2 respectively, with lateral flows denoted by Q_3 and Q_4. Other associated data include the inflow angles of approach, θ_1, θ_3, θ_4, the structure length, L, the pipe diameters, D_1, D_2, D_3, D_4, and the flowline elevations E_1, E_2, E_3, E_4. Given these data, an estimate of h_j is computed by a steady-flow type formula

$$h_j = f(Q_1, Q_2, Q_3, Q_4, \theta_1, \theta_3, \theta_4, L, D_1, D_2, D_3, D_4, E_1, E_2, E_3, E_4, n) \qquad (1)$$

where n is a Manning's friction constant. One such formula is the widely used Thomson's Equation (see Los Angeles County Design Manual, 1978) which estimates the change in hydraulic-grade-line (HGL), by

$$h_j = \frac{(Q_2V_2 - Q_1V_1 \cos\theta_1 - Q_3V_3 \cos\theta_3 - Q_4V_4 \cos\theta_4)}{\frac{1}{2}g\,(A_1 + A_2)} \qquad (2)$$

which is currently used in many civil engineering software packages for storm drain system hydraulic analysis.

In Eq. (2), V_i are steady flow velocities, g is gravity, and A_1 and A_2 are the flow areas for Q_1 and Q_2, respectively. Whitley and Hromadka (1990) developed an analytic solution, and computer program, for the pressure-plus-momentum balance analysis in pipeflow junction structures, and found Eq. (2) to be a good estimator for a wide range of flows and conditions.

At issue are the values of inflow used in Eq. (1). That is, Q_1 and Q_2 are obtained from a hydrologic analysis of the catchment, but Q_3 and Q_4 are typically chosen simply so that mass continuity is obtained. It is implicitly being assumed that the hydraulic gradeline is maximum when Q_1 and Q_2 values occur. But the value for h_j may not necessarily be maximum when the upstream and downstream mainline flows are Q_1 and Q_2.

To examine the sensitivity of h_j to the choice of junction inflows, the following assumptions are made:

1. Continuity of mass applies.

2. h_j is computable using Eq. (2), where all flows are steady for small durations of time.

PROCEDURE

Let the design peak flows be noted by the values q_1, q_2, q_3, q_4, where q_i is associated to the value Q_i used in Fig. 1. Necessarily, $q_3 \geq Q_3$, $q_4 \geq Q_4$, $q_1 = Q_1$ and $q_2 = Q_2$ are the values generally used in computing h_j. But the lateral peak flows of q_3 and q_4 are generally larger than the values Q_3 and Q_4 used in the estimation of h_j; this is due to the unsteady flow that actually occurs at the junction structure. (It is noted that friction losses, and other energy losses for the mainline, are maximized using the maximum flow-rate such as is used in normal hydraulic analysis procedures; only the junction structure analysis for h_j is subject to the issue of additions to the flow value).

In order to compute h_j, values of structure inflows are needed; i.e., Q_1, Q_3 and Q_4 values are needed where $Q_2 = Q_1 + Q_3 + Q_4$.

The value of Q_2 is determined from hydrologic analysis of the catchment where a type of confluence formula is used at the junction of the three flow streams. For example, one such confluence formula is based upon the stream time-of-concentration T_c values (i.e., T_1, T_3, T_4) and rainfall intensity values corresponding to the T_c values (i.e., I_1, I_3, I_4) and determines an estimate $\hat{Q}_2$ by

$$\hat{Q}_2(t) = \sum_{i=1,3,4} q_i \min\left(\frac{t}{T_i}, \frac{I(t)}{I_i}\right), t > 0 \tag{3}$$

where $I(t)$ is the selected return frequency rainfall intensity value corresponding to duration, t, and the q_i are the hydrologic peak flow values for stream i. In using Eq. (3), $\hat{Q}_2(t)$ is evaluated at time values of $t = T_1, T_3, T_4$, and then Q_2 is chosen as the maximum value obtained (Hromadka et al, 1987):

$$Q_2 = \max\{\hat{Q}_2(t); t = T_1, T_2, T_3\} \tag{4}$$

For long times-of-concentration in the mainline, the lateral inflow may be negligible, which when used in the computation of h_j, may not be the most critical test for the junction structure. Other confluence formulae are discussed in Hromadka et al (1987).

In order to develop estimates of h_j for various inflow combinations, it is assumed that the inflows Q_i are probabilistic, whereas the sum of the Q_i is known by definition of the value $Q_2 = q_2$ (if a smaller value of q_2 is used, then one does not have the desired return frequency design condition outflow, but a lesser return frequency).

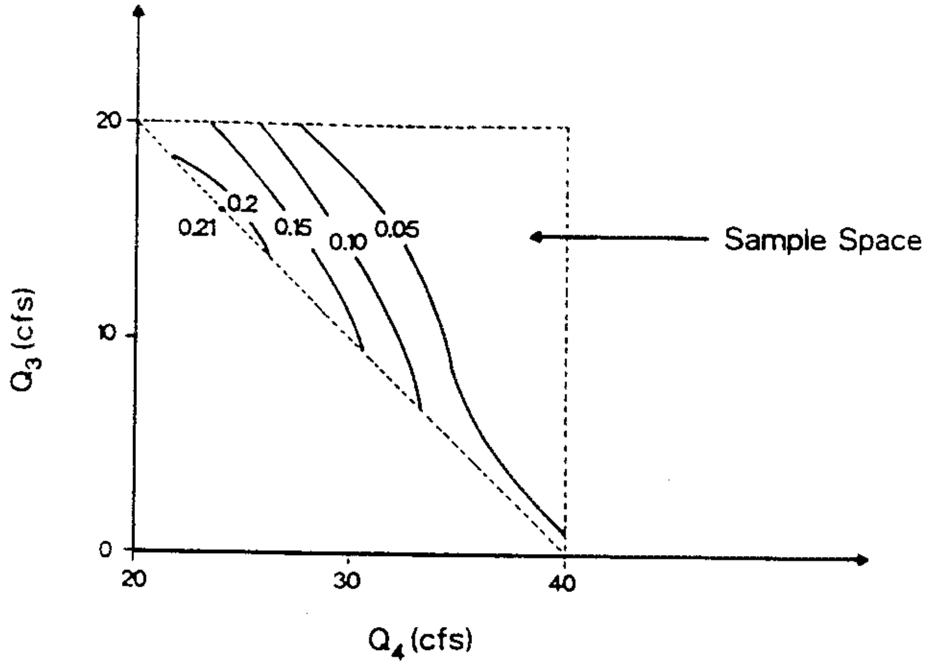

Figure 2. hj Value Contours

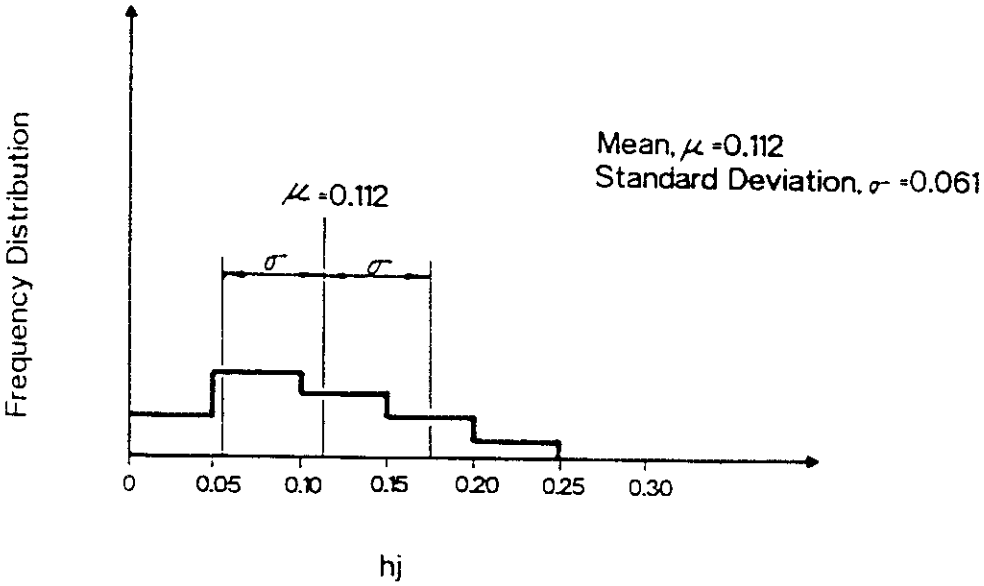

Figure 3. hj Frequency Distribution

The probabilistic distribution of Q_3, Q_4 are assumed to be uniform by (where necessarily $q_3 + q_4 < q_2$)

$$Q_3 = U\,[0, q_3]$$
$$Q_4 = U\,[0, q_4] \tag{5}$$

and, $Q_1 = q_2 - (Q_3 + Q_4)$. Sampling is conditioned such that $0 < Q_1 \leq q_1$; that is, the sampling set of flowrates are rejected unless continuity is satisfied.

The distribution of h_j values is determined by partitioning the Q_3 and Q_4 distributions into a frequency-distribution of values, and exhausting all combinations of (Q_1, Q_3, Q_4) for use in computing h_j values.

COMPUTER PROGRAM

A simple probabilistic model which implemented the above procedures was developed. The program calls the analytic solution pressure-plus-momentum program of Whitley and Hromadka (1990). Input requirements are given below:

Table 1.
PROGRAM INPUT REQUIREMENTS

Variable Name	Description
q_1	Hydrologic Peak Flow for Line #1 (Upstream)
q_2	Hydrologic Peak Flow for Line #2 (Downstream)
q_3	Hydrologic Peak Flow for Line #3 (Lateral)
q_4	Hydrologic Peak Flow for Line #4 (Lateral)
D_1	Upstream Pipe Diameter
D_2	Downstream Pipe Diameter
ΔE	Difference in elevation (i.e., $E_2 - E_1$)
M	Partition size for Probabilistic Analysis

APPLICATION

To demonstrate above procedures, an application is presented using the data of Table 2.

Table 2.
APPLICATION DATA

Variable	Value
q_1	100
q_2	140
q_3	20
q_4	40
θ_1	10°
θ_3	45°
θ_4	30°
D_1	48-inch
D_2	60-inch
D_3	18-inch
D_4	24-inch
E_1	100.0
E_2	99.0
E_3	99.5
E_4	99.5
L	15
n	.013
S_1	.0050
S_2	.0045
S_3	.0032
S_4	.0028
m	8

For the problem of Table 2, h_j values where computed according to the uniform distribution of lateral flow values of Eq. (5), and the continuity equation. Contours of computed h_j values are shown in Fig. 2. A frequency distribution of h_j values is shown in Fig. 3. From Fig. 2, a maximum value of h_j is 0.210, whereas a mean value is 0.112 and standard deviation of h_j is 0.061. The chosen value of h_j would then be used as the design junction energy loss for the hydraulic analysis.

CONCLUSIONS

A junction structure analysis computer program is developed for use in civil engineering design of storm drain pipe systems. This program is novel because it enables the engineer to evaluate combinations of quasi steady-state flow regimes within the junction structure. The program utilizes an analytic solution of the pressure-plus-momentum balance equation for computing energy loss values. Because storm drain systems are designed to accommodate maximum test flow conditions, use of the proposed approach will enable a more consistent design than given by the usual single analysis methods in common use. The application demonstrates the utility of the proposed model.

REFERENCES

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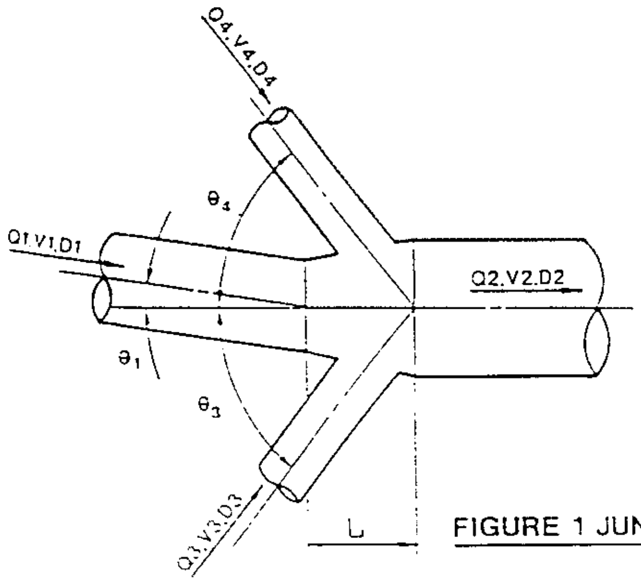

FIGURE 1 JUNCTION LOSS MODEL GEOMETRY