Session MA2.2 Time Series Analysis II, Room AL 113 ... Page 97
Chair: A.H. El-Shaarawi

- *Induced Stochastic Models in Hydrology* by R.W. Katz and M.B. Parlange
- *Application of Transfer Function Model on Storage-Runoff Process* by P.-S. Yu, C.-L. Liu, and T.-Y. Lee
- *Linear Procedures for Time Series Models in Hydrology* by P.R. de Holanda Sales, B. de Bragança Pereira, and A. de Mesquita Vieira
- *Optimal Parameter Estimation of Conceptually-Based Streamflow Models by Time Series Aggregation* by P. Claps and F. Murrone

Session MA2.3 Stochastic Optimization II, Room AL 105 ... Page 107
Session Organizers and Chairs: A. Bogobowicz and K. Ponnamalam

- *A Comparison of Stochastic Optimization Methods for Multi-Reservoir Systems* by M. Bessa, S. Fletcher, K. Ponnamalam, and W.F. Curi
- *Stochastic Approach to an Idle Discharge from Energy Reservoirs* by W. Indyk and A. Potocki
- *Estimation of Parameters of the Quasi-Distributed Catchment Runoff Model Topmodel Using Generalised Likelihood Uncertainty Estimation on a UK Catchment* by R. Romanowicz and K. Beven

Session MA2.4 Decision Making and Decision Support II, Room AL 124 ... Page 117
Chair: K. Kobayashi

- *Knowledge Acquisition and Qualitative Reasoning for Flood Control* by S. Oishi and S. Ikebuchi
- *The Impact of Stochastic Decision Criteria on the Ranking of Farm Management Alternatives by a USDA-ARS Decision Support System* by D.S. Yakowitz, B. Imam, and L.J. Late
- *Decision Support System for Multi-Reservoir Operation* by H. Raman, N. S. Kumar, and D. L. Mastrod

Session MA2.5 Geomorphology, Room AL 206 ... Page 127
Chair: R. Soulsi

- *Evaluating the Effect of Land Development on Sediment Transport Using a Probability Density Function* by T.V. Hromadka II and R.J. Whitley
- *Basin Scale for Distributed Runoff Model Based on Channel Network Geometry* by H. Hayakawa and M. Fujita
- *Runoff Analysis by the Quasi Channel Network Model in the Toyohira River Basin* by H. Saga, T. Nishimura, and M. Fujita
HYDRAULIC JUNCTION PROBABILISTIC ANALYSIS FOR A HYDROLOGIC CONFLUENCE

Theodore V. Hromadka II and Chung-Cheng Yen
Boyle Engineering Corporation
1501 Quail Street
Newport Beach, CA 92658-9020

Robert J. Whitley
Department of Mathematics
University of California
Irvine, CA 92717

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 straightforward 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, \( \theta_1, \theta_3, \theta_4 \), 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

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

(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

\[
    h_j = \frac{(Q_2V_2 - Q_1V_1 \cos \theta_1 - Q_3V_3 \cos \theta_3 - Q_4V_4 \cos \theta_4)}{2g(A_1 + A_2)}
\]

(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 grade-line 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

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

(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):

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

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

\[
Q_3 = U[0, q_3] \\
Q_4 = U[0, q_4]
\]

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 $h_i$ 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 $h_i$ 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>
<thead>
<tr>
<th>Variable Name</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_1$</td>
<td>Hydrologic Peak Flow for Line #1 (Upstream)</td>
</tr>
<tr>
<td>$q_2$</td>
<td>Hydrologic Peak Flow for Line #2 (Downstream)</td>
</tr>
<tr>
<td>$q_3$</td>
<td>Hydrologic Peak Flow for Line #3 (Lateral)</td>
</tr>
<tr>
<td>$q_4$</td>
<td>Hydrologic Peak Flow for Line #4 (Lateral)</td>
</tr>
<tr>
<td>$D_1$</td>
<td>Upstream Pipe Diameter</td>
</tr>
<tr>
<td>$D_2$</td>
<td>Downstream Pipe Diameter</td>
</tr>
<tr>
<td>$\Delta E$</td>
<td>Difference in elevation (i.e., $E_2 - E_1$)</td>
</tr>
<tr>
<td>$M$</td>
<td>Partition size for Probabilistic Analysis</td>
</tr>
</tbody>
</table>
APPLICATION

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

<table>
<thead>
<tr>
<th>Variable</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_1$</td>
<td>100</td>
</tr>
<tr>
<td>$q_2$</td>
<td>140</td>
</tr>
<tr>
<td>$q_3$</td>
<td>20</td>
</tr>
<tr>
<td>$q_4$</td>
<td>40</td>
</tr>
<tr>
<td>$\theta_1$</td>
<td>10°</td>
</tr>
<tr>
<td>$\theta_3$</td>
<td>45°</td>
</tr>
<tr>
<td>$\theta_4$</td>
<td>30°</td>
</tr>
<tr>
<td>$D_1$</td>
<td>48-inch</td>
</tr>
<tr>
<td>$D_2$</td>
<td>60-inch</td>
</tr>
<tr>
<td>$D_3$</td>
<td>18-inch</td>
</tr>
<tr>
<td>$D_4$</td>
<td>24-inch</td>
</tr>
<tr>
<td>$E_1$</td>
<td>100.0</td>
</tr>
<tr>
<td>$E_2$</td>
<td>99.0</td>
</tr>
<tr>
<td>$E_3$</td>
<td>99.5</td>
</tr>
<tr>
<td>$E_4$</td>
<td>99.5</td>
</tr>
<tr>
<td>$L$</td>
<td>15</td>
</tr>
<tr>
<td>$n$</td>
<td>0.013</td>
</tr>
<tr>
<td>$S_1$</td>
<td>0.0050</td>
</tr>
<tr>
<td>$S_2$</td>
<td>0.0045</td>
</tr>
<tr>
<td>$S_3$</td>
<td>0.0032</td>
</tr>
<tr>
<td>$S_4$</td>
<td>0.0028</td>
</tr>
<tr>
<td>$m$</td>
<td>8</td>
</tr>
</tbody>
</table>
For the problem of Table 2, $h_j$ values were 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


Figure 2. hj Value Contours

Figure 3. hj Frequency Distribution