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Yeou-Koung Tung　　Yixing Bao

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Yeou-Koung Tung
Wyoming Water Research Center
and
Department of Statistics
University of Wyoming
Laramie, Wyoming

Yixing Bao
Environmental System Sec.
Texas Water Development Board
Austin, Texas

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OPTIMAL DESIGN OF HIGHWAY DRAINAGE STRUCTURES

Yeou-Koung Tung[1], A.M. and Yixing Bao[2]

ABSTRACT

Hydraulic design of a bridge or culvert using a risk-based approach is to choose among the alternatives the one associated with the least total expected cost. In this paper, the risk-based design procedure is applied to pipe culvert design. The effect of the hydrologic uncertainties such as sample size and type of distribution model on the optimal culvert design parameters including design return period and total expected cost are examined in this paper.

INTRODUCTION

The basic functions of highway drainage structures are: (1) as hydraulic facilities to safely convey floods across highways under all but severe flooding conditions, and (2) as portions of the highway to move highway traffic freely over stream channels. There are two general types of drainage structures: bridges and culverts.

The design of highway drainage structures involves both hydraulic design and structural design. Hydraulic design of a highway drainage structure consists of analyses of the hydraulic performance of the structure in conveying flood water across the roadway and determinations of the most economical design alternative. The investment cost is dependent on the environmental conditions such as the location of the structure, geomorphic conditions, the soil type at the structure site, type and price of construction material, hydraulic conditions such as slope of the stream bottom, and flow conditions, recovery factor of the capital investment, labor and transportation costs.

In reality, the investment cost also involves various uncertainties. However, the consideration of the uncertainties in the investment cost is beyond the scope of this paper.

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[1] Assoc. Prof., Wyoming Wat. Resear. Ctr. and Statis. Dept., Univ. of Wy., Laramie, WY 82071

[2] Hydrol., Environ. Syst. Sec., Texas Wat. Devel. Board, Austin, TX 78711

OPTIMAL RISK-BASED DESIGN PROCEDURE

Because the risk cost associated with the failure of hydraulic structure can not be predicted from year to year, a practical way is to quantify it using an expected value on the annual basis. The annual total expected cost (ATEC) associated with a highway drainage structure is the sum of the annual installation cost and annual expected flood damage cost which can be expressed as

$$ATEC(q_c) = FC(q_c)*CRF + E(D|q_c) \tag{1}$$

in which FC is the first or total installation costs, CRF is the capital recovery factor, and E(D) is the annual expected damage cost (the second cost).

Mathematically, the optimal risk-based design problem can be stated as:

$$\text{Minimize} \quad ATEC(q_c) = FC(q_c)*CRF + E(D|q_c) \tag{2}$$

subject to

$$q_{min} \le q_c \le q_{max}. \tag{3}$$

EVALUATIONS OF ANNUAL EXPECTED FLOOD DAMAGE COST

An important task in risk-based design is to evaluate the annual expected damage cost. The conventional risk-based design computes the annual expected damage by

$$E(D|q_c) = \int_{q_c}^{\infty} D(q|q_c)\ f(q)\ dq \tag{4}$$

where q_c is the flow capacity of a hydraulic structure subject to random flood loadings following a probability density function (PDF) of f(q) and $D(q|q_c)$ is the damage function.

Note that Eq.(4) considers only the inherent hydrologic uncertainty due to the random occurrences of flood event. It does not consider hydraulic and economic uncertainties. A perfect knowledge about the probability distribution of flood flow is assumed. This is generally not the case in reality.

Since the occurrence of streamflow is random, the statistical properties such as the mean, standard deviation and skewness of the distribution calculated from a finite sample are also random. Therefore, the flood magnitude of a given return period calculated from sample statistics is also a random variable associated with its probability distribution instead of being single-valued presented by its "average" as commonly done in practice.

To combine the hydrological inherent and parameter uncertainties, the annual expected damage cost can be written as

$$E(D|q_c) = \int_{q_c}^{\infty}\left[\int_{q_c}^{\infty} D(q_{TR})\ h(q_{TR})\ dq_{TR}\right] f(q)\ dq \qquad (5)$$

in which $h(q_{TR})$ is the PDF of Q_{TR}. An investigation on the effect of sample size and distribution type on the annual expected damage cost is made by Bao et al. (1987).

SAMPLING PDF OF FLOOD MAGNITUDE ESTIMATOR

Computing the annual expected flood damage using Eq.(5) requires the distribution of the flood magnitude estimator Q_{TR} which is estimated as

$$Q_{TR} = \bar{Q} + K_{TR}\ S \qquad (6)$$

in which $\bar{Q}$ and S are the sample estimators of the mean and standard deviation, respectively, and K_{TR} is the frequency factor for a TR-year event. Q_{TR} is a random variable resulting from the combined effects of all hydrologic parameter uncertainties involved in $\bar{Q}$, S, and K_{TR}. Therefore, for a given return period, Q_{TR} is associated with a probability density function $h(q_{TR})$.

For a normal population, Johnson (1940) showed that $\sqrt{n}(Q_{TR} - \bar{Q})/S$ has a noncentral t-distribution with a noncentrality parameter $\delta = z_{TR}\sqrt{n}$ with (n-1) degree of freedom, where z_{TR} is the (1-1/TR)-th quantile of the standard normal random variable and n is the record length. For Pearson type III or log-Pearson type III distributions, Stedinger (1983) derived an expression to approximate the quantile of the distribution of Q_{TR}.

APPLICATION TO OPTIMAL PIPE CULVERT DESIGN

The problem is to design a circular culvert under a two-lane highway. The data used in this numerical example are from Corry et al. (1980). The example aims at investigating the sensitivity of the optimal design parameters to (1) the hydrologic parameter uncertainty, (2) the length of streamflow records, and (3) the distribution model of flood flow. Uncertainties in hydraulic, structural, and economic aspects are not considered.

The estimated sample mean and sample standard deviation for the flood flow are 47.9 and 71.9 cfs, respectively. The skew coefficient of streamflow for the original scale and log-transformed scale are assumed to be 0.5 and 0.2, respectively. The damage function D(q) used is

$$D(q,\ q_c) = \begin{cases} D_{max}\left(\dfrac{q - q_c}{q_{max} - q_c}\right), & q_c \le q \le q_{max} \\ D_{max}, & q \ge q_{max} \\ 0 & q \le q_c \end{cases} \qquad (7)$$

in which q_{max} is the flood magnitude corresponding to the maximum damage D_{max}.

Because of the complexity of the objective function, analytical solution to the optimal culvert design problem is difficult. Therefore, optimum search technique using Fibonacci search is efficient for this single-decision variable optimization problem (Sivazlian and Stanfel, 1974).

RESULTS AND DISCUSSION

With D_{max}=\$928, the optimal design frequency (TR^*), the associated least annual total expected cost (LATEC), the optimal annual first cost (FC^*), and the optimal annual expected flood damage cost (SC^*) for different record length (n) and streamflow probability distributions are shown in Table 1. The values in the columns for n=5~100 are calculated by considering hydrologic parameter uncertainty, while the values in the column with n=∞ were calculated without considering hydrologic parameter uncertainty.

It is observed that when the record length is short, say n<40, the optimal second cost decreases as the record length increases but the optimal first cost increases for a longer record length. The behavior resulting from varying the optimal first or second cost is primarily determined by the geometry features of the FC and SC curves.

Comparing the two design methods, the value of the LATEC without considering parameter uncertainty is always smaller than the one with considering parameter uncertainty. This shows that the negligence of the hydrologic parameter uncertainty could lead to underestimation of the total expected cost.

TABLE 1 OPTIMAL TR , FC , SC AND LATEC FOR DIFFERENT DISTRIBUTIONS AND RECORD LENGTHS, WHEN Dmax = $928

		Record Length (Years)						
		5	10	20	40	60	100	∞
N+	TR* (yr)	4.0	4.82	4.62	4.55	4.55	4.20	4.00
	FC* ($)	377.7	390.0	387.0	386.0	386.0	380.9	377.7
	SC* ($)	117.7	83.5	74.7	70.2	68.7	72.6	70.4
	LATEC ($)	495.4	473.5	461.7	456.2	454.7	453.5	448.1
LN+	TR* (yr)	5.51	5.79	6.0	6.27	6.55	6.20	6.96
	FC* ($)	355.0	357.0	358.4	360.5	362.5	360.0	365.6
	SC* ($)	116.1	89.3	75.5	67.6	63.4	63.3	54.6
	LATEC ($)	471.1	446.7	433.9	428.1	425.9	423.2	420.2
P+	TR* (yr)	4.27	4.62	4.62	4.55	4.55	4.55	4.00
	FC* ($)	378.0	383.3	383.3	382.3	382.3	382.3	373.5
	SC* ($)	122.9	96.5	84.9	80.3	78.7	77.6	80.5
	LATEC ($)	500.9	479.9	468.3	462.6	461.0	459.8	454.1
LP+	TR* (yr)	5.31	5.79	6.00	6.20	6.41	6.13	6.68
	FC* ($)	353.1	356.5	358.2	359.3	361.4	359.2	363.6
	SC* ($)	121.2	93.9	79.8	72.5	68.8	68.2	61.0
	LATEC ($)	474.3	450.5	438.0	432.3	430.2	427.4	424.6

+: N = Normal, LN = Log-Normal
P = Pearson type III; LP = log-Pearson type III.

The value of LATEC decreases as the record length increases. This is expected since the effect of hydrologic parameter uncertainty involved in estimating the second cost diminishes as the record length for streamflow gets longer. The difference in LATEC values calculated by the two methods, with n>20, is only about 3% for any of the four probability distributions considered.

Examining the TR* values in Table 1, the difference in TR* between the two methods is less than 20% in most cases. However, there does not exist the same consistent tendency as with the LATEC discussed above. Furthermore, the value of TR* fluctuates as the record length increases, although in most cases, when the record length is longer than 60 years, the difference in TR* between the two methods becomes smaller. Therefore, when TR* is considered as a criterion in the comparison of the two design methods, it is difficult to conclude which method tends to be more conservative.

APPENDIX I - References

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