A METHOD FOR OPTIMUM LAYOUT DESIGN OF CONCRETE GRAVITY DAMS

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Abstract: A computer-assisted desktop method is developed for layout design of a concrete gravity dam on the basis of safety and economy. Using a set of regression equations, a dam layout is proposed. With reference to the regression equations and relevant input data, overall dam dimensions are determined by satisfying the stability criteria jointly under usual, unusual, and extreme loading conditions with the desired hydraulic conformity. Among several feasible alternatives, the program enables a designer to select the optimum layout, which corresponds to the minimum total cost of the structure. The method is applied to a case study to examine dimensions of proposed alternatives and to compare them with those of an existing dam.

Key Words: Concrete gravity dam, layout design, spillway, safety, load combinations

1. INTRODUCTION

A concrete gravity dam is primarily designed to offer sufficient resistance against all disturbing agents exerted upon it by its own weight. A designer tackles with the problem of selection of a suitable layout for a gravity dam, which can be constructed at a site where the foundation is composed of sound rock. Uncertainties in gravity dam design may pose a potential hazard to the human life and property in close vicinity. To offset the risk to a certain extent, a conservative design may be performed to have sufficient resistance against all possible disturbing tendencies. A computer-assisted method is developed for optimum layout design of a concrete gravity dam on the basis of cost minimization with desired hydraulic conformity and structural safety under all possible loading combinations. Use of the method enables a designer to compare a number of layout alternatives for a gravity dam, which are feasible in terms of hydraulic, structural, and economic evaluations. The structural performance of the selected layout is examined based on a gravity analysis using a user-friendly and flexible computer program. The program is illustrated in an example.

2. DEVELOPMENT OF THE METHOD

A computer program is developed in MS-Fortran programming language to perform an optimum layout design for a concrete gravity dam. A model cross-sectional layout is proposed for a concrete gravity dam (See Fig. 1). By examining geometric features of some existing multi-purpose concrete gravity dams throughout the world, Yanmaz et al. (1999) proposed the following regression equations to define the shape and size of a gravity dam:

$$H_t = 1.0711\, H_n \quad (1)$$

$$H_{sc} = 0.9086\, H_n \quad (2)$$

$$H_m = 1.0529 H_n \quad (3)$$

$$H^* = 0.1075 H_t \quad (4)$$

$$t_c = 0.0483 H_m + 2.392 \quad (5)$$

in which all dimensions are in m.

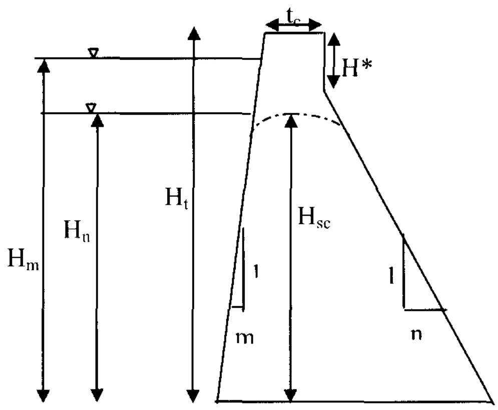

Fig. 1. Cross-sectional Layout of the Concrete Gravity Dam

The initial water depth in the reservoir is assumed as the normal operating depth, which can be estimated according to the project requirements of the dam. Although Equations (1) through (5) have been obtained from the geometric correlations of some existing multi-purpose dams, these equations may be used for the gravity dam design. This is mainly due to the fact that the designer is supposed to initiate the program with the assumed normal operating level, which reflects the purpose of the reservoir operation and topographic characteristics of the reservoir. With the height of the overflow spillway obtained from Equation (2), successive spillway length values are tested in an iterative scheme in 0.1 m intervals between the minimum and maximum allowable values, which are dictated by the local topographic conditions. The reservoir routing calculations are carried out for various spillway lengths such that the maximum reservoir depth under the design conditions is coincident with that obtained from Equation (3). Use of Equation (3) gives a guide to select a reasonable value for the length of the overflow spillway. Similar computations are repeated for the successive increments of the spillway crest height from its initial value to a maximum specified value in 0.2 m increments of height. For each step, the type of USBR stilling basin is determined using the energy equation between the upstream and the toe of the spillway. The bed elevation of the stilling basin is assumed to be the same as the thalweg elevation at the upstream. The continuity equation to be used in routing can be expressed as:

$$\frac{dh}{dt} = \frac{I(t) - Q(h)}{A(h)} \quad (6)$$

where h is the height measured from the reservoir bed, t is the time, I is the inflow, Q is the outflow, and A is the reservoir surface area. For ungaged basins, the inflow hydrograph is to be modeled. The following inflow hydrograph (Horn, 1987) is used in the method for engaged basins which is obtained by fitting the dimensionless unit hydrograph of US Soil Conservation Service (Chow et al., 1988) in the general form of a Pearson type 3 probability density function

$$I(t) = I_p \left(\frac{t}{t_p} \right)^{3.5} \exp\left(-3.5 \left(\frac{t}{t_p} - 1 \right) \right) \quad (7)$$

in which I_p and t_p are peak inflow and time to peak, respectively that can be determined synthetically according to the hydrological characteristics of the basin. By inserting the mathematical expressions for I(t), Q(h) and A(h) into Equation (6), a first order non-linear ordinary differential equation is obtained which can be solved using the fourth order Runge-Kutta method. In the routing, all types of outlet works except for the overflow spillway are accepted closed. Through the execution of the first step of the program, a number of combinations for the spillway heights and corresponding crest lengths are obtained for which the outflows leaving the reservoir are smaller than the maximum allowable outflow in order not to cause serious inundation problems at the downstream.

For a particular combination of the spillway crest height and length, the other characteristic dimensions of the dam as shown in Fig. 1, are determined using Equations (1), (4) and (5). The program considers various side inclination values ranging from m=0.0 to 1.0 and n=0.0 to 2.0 where m and n are the horizontal values of the side inclinations, respectively. Because of structural safety requirements, the value of n is taken to be greater than that of m. A conservative design is proposed in this method since it is based on the satisfaction the safety criteria under all possible loading combinations simultaneously. The following loading criteria may be used as a general guideline (USBR, 1976 and 1987). Usual loading consists of hydrostatic force produced by a reservoir at normal operating level; uplift force; temperature stresses produced by temperatures normal for that time of the year; dead loads; ice load and silt load. In unusual loading, hydrostatic force produced by a reservoir at full upstream level; uplift force; stresses produced by minimum temperature at full level; dead loads and silt load are considered. Extreme loading consists of forces in usual loading and earthquake forces.

The ability of a concrete gravity dam to resist the applied loads is measured by the safety factors against overturning, FS_o; sliding, FS_s; and combined shear and sliding, FS_{ss}. The overall safety of the dam may be assessed with reference to these safety factors computed at the foundation level where maximum disturbing tendencies develop. The contact stress between the foundation and the dam and internal stresses in the dam body must be compressive. The maximum compressive stress, σ_{max}, must be smaller than a certain fraction of the allowable compressive stress for concrete, σ_{ac}, and foundation material, σ_{af} (Table 1).

The program is executed for various combinations of side inclinations. Finally, a list is formed which presents a set of feasible alternatives for the proposed dam layout with the corresponding safety factors, pressures, shear stresses and total costs. For each successive trial the dam stability is assessed according to the

Table 1. Safety Criteria for Concrete Gravity Dams (USBR, 1976 and 1987; Thomas, 1976).

Loading	FS_s	FS_o	FS_{ss}	σ_{max} in concrete	σ_{max} In foundation
Usual	≥ 1.5	≥ 2.0	≥ 5.0	$\leq \sigma_{ac}/3.0$	$\leq \sigma_{af}/4.0$
Unusual	≥ 1.2	≥ 1.5	≥ 4.0	$\leq \sigma_{ac}/2.0$	$\leq \sigma_{af}/2.7$
Extreme	> 1.0	≥ 1.2	≥ 3.0	$\leq \sigma_{ac}$	$\leq \sigma_{af}/1.3$

safety criteria given in Table 1 under usual, unusual, and extreme loading cases jointly. The vertical normal stresses, principal stresses parallel to the faces of the dam, and internal shear stresses in the dam are also computed.

3. COST COMPUTATIONS

The total cost of the structure is accepted as the summation of costs of the dam body, overflow spillway, energy dissipating basin, and relevant appurtenances. The other possible costs, e.g., costs of expropriation, excavation, water intake structure, and site installations are ignored since they will be approximately the same for each alternative. With reference to the practical applications concerning dam constructions in Turkey, the 2000-year unit price of concrete in the dam body is taken as \$87.4/m^3 which includes the costs of admixtures used for plain concrete, formwork and scaffolding, workmanship and transportation. Unit cost of concrete in the overflow spillway and stilling basin is accepted as \$132.9/m^3 including the costs of admixtures for reinforced concrete and other items as considered in the unit price of the dam body.

The authors propose the following equation for the volume of the spillway, V_s, according to the standard vertical-face spillway crest profile (USBR, 1987) that is composed of two curvatures at the upstream face and has a parabolic relation at the downstream face.

$$V_s = L\begin{pmatrix} 0.02279H_0^2 + 0.282H_0(H_{sc} - 0.1495H_0) \\ + H_{sc}X_{sc} - \dfrac{K}{\left((n_p+1)H_0^{n_p-1}\right)}X_{sc}^{n_p+1} \end{pmatrix} \tag{12}$$

where L is the length of the overflow spillway; H_0 is the total design spillway head; X_{sc} is the base width of the spillway, which can be determined from the parabolic relation proposed for the downstream face of the spillway (USBR, 1987); and K and n_p are the coefficient and power of the parabolic relation. The following best-fit equations are derived for K and n_p from the graphical presentation given in USBR (1987):

$$K = -2.6335\left(\frac{h_a}{H_0}\right)^2 0.384\left(\frac{h_a}{H_0}\right) + 0.4981 \tag{13}$$

$$n_p = 2.379\left(\frac{h_a}{H_0}\right)^2 - 0.6545\left(\frac{h_a}{H_0}\right) + 1.8756 \tag{14}$$

in which h_a is the approach velocity head. With reference to the layout of the proposed stilling basins (USBR types 1, 2, 3, and 4), which can be found in USBR (1987), the authors developed the following expressions; V_{sb1}, V_{sb2}, V_{sb3}, and V_{sb4}, for the volumes of the USBR stilling basins of types 1, 2, 3, and 4 for one-meter of slab thickness, respectively.

$$V_{sb1} = Ly_1\left(-0.1521F_{r1}^2 + 11.487F_{r1} - 12.107\right) \tag{15}$$

$$V_{sb2} = L\left(\frac{y_1^2}{4.0} + 0.0022y_2^2 + 4.3y_2\right) \tag{16}$$

$$V_{sb3} = L\begin{pmatrix} \dfrac{y_1^2}{4.0} + 0.00972y_1^2(4.0 + F_{r1})^2 \\ + 0.0123y_1^2(9.0 + F_{r1})^2 + 2.7y_2 \end{pmatrix} \tag{17}$$

$$V_{sb4} = L\left(\frac{y_1^2}{1.75} + 0.0123y_1^2(9.0 + F_{r1})^2 + 6.1y_2\right) \tag{18}$$

where y_1 and y_2 are the initial and sequent depths of the hydraulic jump at the stilling basin, respectively, and F_{r1} is the flow Froude number at the toe of the spillway having a rectangular cross-section. For concrete gravity dams having considerably large heights and stilling basin lengths, the value of the uplift force acting beneath the stilling basin may reach to very big values. So, the corresponding slab thickness to overcome the uplift force becomes unrealistically large which may indicate a considerable increase in the cost of the stilling basin. Therefore, one-meter slab thickness is assumed for the stilling basin to carry out the relative comparisons and the corresponding anchorage requirement to withstand the difference between the slab weight and uplift force is estimated in terms of cost. Using 2000-year unit prices accepted by Turkish State Hydraulic Works, the cost of drilling for anchorage is taken as \$29.33/m for moderately stiff foundation conditions. The unit cost of the reinforcing bar, which has a typical diameter of 26 mm, and its placement is taken as \$0.69/kgf. The magnitude of the anchorage force is computed such that it overcomes the net vertical force, i.e. uplift force minus weight of the slab. Using the formulas proposed in the manuals of USBR and Davis and Sorenson (1969), the weight of a radial gate, W_r in kgf, is determined from

$$W_r = 0.0535 h_g^{3.25} \tag{19}$$

where h_g is the height of the gate that is taken as the vertical distance between the crest elevagency gate, W_e in kgf, which is installed in front of the radial gate, is determined from

$$W_e = 0.0554 h_g^{3.25} \tag{20}$$

The weight of the lifting mechanism used for the radial gate, W_l in kgf, is obtained from

$$W_l = 0.0434 h_g^{3.25} \tag{21}$$

In the determination of the unit costs of gates including the installation, 2000-year unit prices of Turkish State Hydraulic Works are used as \$4.32/kgf for radial gates, \$3.76/kgf for emergency gates, which are of vertical lift type, and \$5.96/kgf for the lifting mechanism of the radial gates.

4. APPLICATION

In the application, the Porsuk Dam, a concrete gravity dam on the Porsuk creek in Turkey, is considered. To make a relative comparison, the input data (Table 2), which reflect the existing design, are taken in the execution of the program.

With the run of the program, an output is obtained which presents sets of feasible alternatives in terms of the height and length of the overflow spillway with the corresponding total costs of the dam for various combinations of the side inclinations of m and n. In the computation of the volume of the dam body, the designer assigns the length of a rectangular valley having an equivalent cross-sectional area as the existing cross-section of the valley. Variation of the total cost of the structure with respect to the m values is shown in Fig. 2 for the Porsuk Dam. As the spillway crest height increases, the reservoir accommodates greater volumes of flood that may require smaller crest lengths, and hence smaller outflows can be released from the reservoir (see Fig. 2). Increase in the spillway crest height leads to an increased volume of the dam body due to the requirement of greater dam

Table 2. Input Data Used in the Application

Parameter	Value
Design value of t_p	90900 s
Design value of I_p	2080 m^3/s
A(h)	$-0.0135h^6+1.7737h^5-77.794h^4+933.53h^3+23568h^2+123765h$
Initial reservoir depth	41.35 m
Allowable outflow	875 m^3/s
Maximum spillway length	50 m
Design tailwater depth	6 m
Average valley width	210 m
Submerged specific weight of sediment	11 kN/m^3
Thickness of ice sheet	m
Rate of increase of temperature	2.8°C/hr
Depth of sediment accumulation	m
Vertical earthquake coefficient	0.04
Horizontal earthquake coefficient	0.08

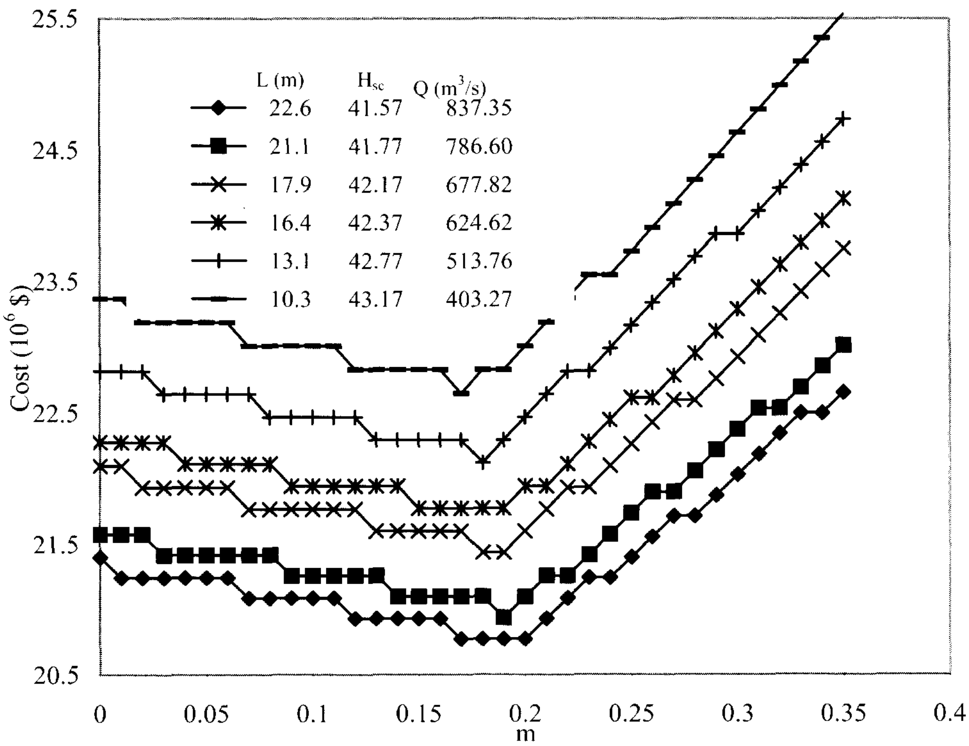

Fig. 2. Relation Between Total Cost and Upstream Side Inclination for Porsuk Dam.

crest height. This also leads to a decrease in the spillway crest length. For this case, the total volume of spillway decreases slightly since the rate of increase of H_{sc} is smaller than the rate of decrease of the spillway crest length. However, as the rate of increase of the volume of the dam body is greater than the rate of decrease of the spillway volume, the total cost of the structure increases with increasing spillway crest height. For a particular combination of L and H_{sc}, the minimum total cost may be achieved under a case having m>0.0 for which smaller dimen-

Table 3. Details of the Optimum Alternatives for the Porsuk Dam

m	n	FS_o			FS_{ss}			σ_{max} kN/m^2		
		U	UN	EX	U	UN	EX	U	UN	EX
1.17	0.75	2.317	2.064	1.887	5.002	4.368	3.856	627.33	720.74	766.74
0.18	0.74	2.316	2.065	1.888	5.011	4.377	3.864	637.46	730.56	776.05
0.19	0.73	2.315	2.065	1.888	5.019	4.385	3.872	647.68	740.49	785.45
0.20	0.72	2.314	2.066	1.889	5.028	4.393	3.881	658.00	750.53	794.95

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0.20	0.72	2.314	2.066	1.889	5.028	4.393	3.881	658.00	750.53	794.95

Table 4. Comparison of Dimensions of Existing and Proposed Dams

Alternative	m	n	H_f (m)	H_{sc} (m)	H^* (m)	t_c (m)	L (m)
Existing	0.00	0.85	49.70	40.95	8.70	4.50	22.00
Proposed	0.20	0.72	49.00	41.57	5.26	4.72	22.60

Table 5. Variation of Safety Factors for the Existing and Proposed Dams

Case	FS_o			FS_{ss}			σ_{iu} kN/m^2			σ_{id} kN/m^2		
	U	UN	EX	U	UN	EX	U	UN	EX	U	UN	EX
Existing	2.13	1.79	1.72	4.45	3.73	4.12	554	714	820	608	294	149
Proposed	2.31	2.07	1.90	5.03	4.39	3.88	658	751	795	347	248	163

sions for the dam can be obtained with the required safety level under all loading conditions (Fig. 2).

As can further be observed from Fig. 2, for a particular value of L, there may be a minimum total cost for more than one feasible alternative. This is due to the fact that an increase in any m value would result in a decrease in the corresponding n value to satisfy the minimum required safety level. Therefore, it is possible to obtain the same volumes and total costs under close ranges of m and n values. The selection of the final dimensions from several feasible alternatives having the same minimum total costs can be done according to the further examination of the safety factors and base pressures which may exhibit slight variations among the loading conditions concerned due to the differences in m and n values. For the Porsuk Dam, 4 alternatives having the minimum total costs under L= 22.60 m are further examined according to the safety factors and base pressures (see Table 3). Therefore, the proposed side inclinations are taken as m=0.20 and n=0.72. The dimensions of the existing and the proposed dams are presented in Table 4. The values of the safety factors and base pressures for the selected alternative are presented in Table 5 for usual (U), unusual (UN) and extreme (EX) loading conditions. As can be observed from Table 5, the existing dam is observed to be critical with respect to the combined shear and sliding under usual and unusual loadings. It should be stated, however, that the concluding remarks are based on the proposed method that involves the satisfaction of the safety under all loading combinations jointly.

5. CONCLUSIONS

A desktop method is developed for optimum layout design of a concrete gravity dam based on cost minimization. The method reduces the computational effort to achieve an optimum solution among various feasible alternatives. It proposes a layout with reference to a set of regression equations, which are derived from the geometric details of some existing multi-purpose gravity dams. The topographic characteristics of the reservoir are considered implicitly in the normal operating level since the information concerning the reservoir elevation-area is available. Furthermore, a reservoir routing is carried out for determining the required flood storage behind the gravity dam using the peak inflow characteristics given by Equation (7) that may be changed by the user according to local hydrologic regime. Therefore, it can be stated that this method may be generalized for the layout design of concrete gravity dams having different purposes and topographical characteristics than those of the calibration data presented in Table 1. The overall dimensions of the dam are obtained on the basis of cost minimization and satisfaction of the stability under usual, unusual, and extreme loading conditions jointly. Among several feasible alternatives, the program enables a designer to select the optimum layout that corresponds to the minimum total cost of the structure. As the optimum operation of a reservoir is controlled by proper operation of the outlet works, the benefits derived from the specified water levels and storage at normal operating level will almost be constant throughout the successive trials for the alternatives. That is why the maximization of the net benefit is not considered for sizing a conservative layout. The method is applied to a case study to examine the dimensions and safety levels of the proposed structure and to compare them with those of the existing structures.

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