STRENGTHENING OF MASONRY DAM BY EARTH BACKFILL

A.R. Chandrasekaran (I)
N.C. Singhal (II)
K.K.M. Menon (III)
Presenting Author : N.C. Singhal

SUMMARY

This paper describes analytical studies carried out to determine the efficacy of strengthening a masonry dam by earth fill placed on the downstream face. The existing masonry dam without the proposed earth fill was first analysed for the self weight, water pressure, uplift and postulated time history of motion using two-dimensional finite element technique. Different profiles of earth backing on the downstream side of the masonry dam were assumed. These consisted of earth backing up to full height, partial height and different downstream slopes. Finite elements were also used to represent the behaviour of earth fill. The results indicate that the stresses in the masonry dam can be reduced by strengthening it with earth fill. The analysis of different profiles of earth backing indicates that an efficient design of strengthening is possible.

INTRODUCTION

In the past, structures were mainly designed based on an adhoc seismic coefficient proposed for the site. In the case of dams the coefficients had values like five, ten or fifteen percent. Seismic zones of a country can be upgraded due to occurrence of new earthquakes like the change made in Indian Code after Koyna earthquake of December 1967. The new criteria developed in U.S.A. for siting nuclear power plants are now increasingly being adopted for relatively less critical but important structures like dams. These methods always give much higher values of earthquake parameters than those given by Codes. Hence, it becomes necessary to check the safety of existing dams for upgraded earthquake intensities. Such an exercise is being carried out for a number of dams in Maharashtra, Western India and it is reported that similar studies are carried out for dams in other countries.

Earth backing is widely accepted form of strengthening of old masonry dams, the designs of which do not conform to the present day criteria, particularly because uplift and earthquake forces were not considered in the past. This method of strengthening has been adopted for many dams in India, like Khadakwasla, Talakdale dams. The earthbacking at Khadakwasla withstood the Koyna earthquake of December 11, 1967 without any manifest external distress (Ref. 1). Khadakwasla is about 145 km north of Koyna. The transverse and vertical components of this accelerogram with a scaling factor of 0.306 have been used in this study.

(I) Professor
(II) Lecturer
(III) Former Graduate Student

Department of Earthquake Engineering,
University of Roorkee,
Roorkee - 247667
INDIA.

For Indian conditions strengthening by earthbacking is very much suited. A gravity dam strengthened by earthbacking functions essentially as a composite gravity dam. The analysis of such a dam is the aim of this paper. Various combinations of earthbacking are tried. For the case of earthbacking, analyses have been made both with and without incorporation of interface elements. In the analysis the dam has been assumed to be fixed at the base, that is, at the level of foundation.

2. DESCRIPTION OF CASES STUDIED

Various cases studied have been divided into following three categories:

(1) Analysis of dam portion only - no fill and no interface elements,

(2) Analysis of dam with backfill, but without interface elements, and

(3) Analysis of dam with backfill and with interface elements.

In case 1, only the dam portion is considered (Fig. 1). In case 2, in addition to the main dam, earthfill is also considered. This case represents the strengthened section of the dam. No interface elements are included in this case. Five alternatives have been considered corresponding to different positions of earth backing as shown in Figs. 2(a) through 2(e). In case 3, interface elements have been introduced between the downstream face of the dam and the fill. Like case 2, here also five cases for different positioning of the backfill have been studied. Fig. 3 shows a typical case with interface elements. In all 11 cases have been studied.

3. METHOD OF ANALYSIS

The finite element method has been used for the analysis. Eight noded isoparametric elements (Ref. 2) have been employed for discretization of the dam and the backfill. The earth backfill represents a case of plane strain and, therefore, the system is analysed as a case of plane strain. Numerical integration at 2x2 Gauss points has been used. The mass matrix has been assumed as diagonal.

The various loads considered for the analysis are, self weight of dam, water pressure and uplift, constituting the static loads, and the earthquake forces. The self weight is represented by equivalent nodal loads due to gravity loading. The water pressure is represented by equivalent nodal loads acting at the interface between water and dam due to normal intensity of water pressure.

The uplift pressure distribution at the contact between dam and foundation is evaluated by assuming a linear pressure distribution of 100% intensity of hydrostatic pressure at upstream, 33⅓% intensity at the line of foundation gallery and zero percent intensity at the downstream face (Ref. 3). The water structure interaction has been represented by a virtual mass of water attached to the upstream face of the dam. The dam has been analysed by simultaneous application of time histories in both horizontal and vertical directions.

The deflections along horizontal and vertical direction at the nodes and the stresses at Gauss integration points have been obtained for various cases due to static and dynamic loads (Ref. 4).

4. INTERFACE ELEMENTS

At the interface of the dam and earthbacking, there will be relative slip and cycles of opening and closing of the interface. The interface behaviour will be nonlinear even under working loads. Conventional finite elements are inadequate to model the interface behaviour. Special elements known as "interface elements" (Ref. 5) have been used in the analysis, the description of which is given below:

Fig. 4 shows a typical six noded isoparametric interface element. The pairs of nodes 1-2, 3-4 and 5-6 are usually close to each other. The coordinates of the middle surface nodes a, b and c at $\eta = 0$, and the normal thicknesses t_a, t_b, and t_c define a variable thickness joint. The thickness, t, at any point is defined by using suitable shape functions $N_i(\xi)$ as

$$t = \sum_{i=1}^{nodes} N_i(\xi) \; t_i = N_a t_a + N_b t_b + N_c t_c \qquad \cdot\cdot\ (1)$$

The relative displacements $\Delta u_a = u_2 - u_1$ and $\Delta v_a = v_2 - v_1$ can be expressed as

$$\{\Delta a\} = \begin{Bmatrix} \Delta u_a \\ \Delta v_a \end{Bmatrix} = \begin{bmatrix} -1 & 0 & 1 & 0 \\ 0 & -1 & 0 & 1 \end{bmatrix} \begin{Bmatrix} u_1 \\ v_1 \\ u_2 \\ v_2 \end{Bmatrix} = [T_a] \; [\delta_a] \qquad \cdot\cdot\ (2)$$

where $\{\delta_a\}$ is the vector of nodal displacements for point a, and $[T_a]$ is an intermediate transfer matrix. The relative displacement of any point can be expressed as

$$\begin{Bmatrix} \Delta u \\ \Delta v \end{Bmatrix} = \begin{bmatrix} N_a & 0 & N_b & 0 & N_c & 0 \\ 0 & N_a & 0 & N_b & 0 & N_c \end{bmatrix} \{\Delta\} = [N]\,[T]\{\delta\} = [N_\delta]\{\delta\} \qquad \cdot\cdot\ (3)$$

where $[N_\delta] = [-I_2 N_a, I_2 N_a, -I_2 N_b, I_2 N_b, -I_2 N_c, I_2 N_c]$ and $[I_2]$ is an identity matrix.

The strain at any point at the interface is defined by local tangential and normal relative displacements $(\Delta u', \Delta v')$ as

$$\begin{Bmatrix} \varepsilon_s \\ \varepsilon_n \end{Bmatrix} = \frac{1}{t} \begin{Bmatrix} \Delta u' \\ \Delta v' \end{Bmatrix} = \frac{1}{t}\; [R] \; [N_\delta]\{\delta\} = [B_i]\{\delta\} \qquad \cdot\cdot\ (4)$$

where ε_s and ε_n are the tangential and normal strains respectively at the point, $[R]$ is the rotation matrix and transfers global strains to local strains. The rotation matrix is derived by the known slope of the curve at the point as

$$[R] = \frac{1}{M} \begin{bmatrix} \frac{dx}{d\xi} & \frac{dy}{d\xi} \\ -\frac{dy}{d\xi} & \frac{dx}{d\xi} \end{bmatrix}$$

$$\text{and } M = \frac{ds}{d\xi} = \left[\left(\frac{dx}{d\xi}\right)^2 + \left(\frac{dy}{d\xi}\right)^2 \right]^{\frac{1}{2}} \quad \text{.. (5)}$$

$[B_i]$ is the strain-displacement matrix of the interface. For interface elements having negligible thickness, t is taken as unity.

The stiffness matrix of the interface element can be derived as

$$[K]^e = \int [B_i][D_i][B_i] \, ds$$

where $[B_i]$ is the strain displacement matrix given by eq.(4), $[D]$ is the elasticity matrix for the interface and ds is a small length of the interface.

The stress at any point can be expressed in terms of strain as

$$\{\sigma\} = \begin{Bmatrix} \tau \\ \sigma_n \end{Bmatrix} = \begin{bmatrix} k_{st} & 0 \\ 0 & k_{nt} \end{bmatrix} \begin{Bmatrix} \epsilon_s \\ \epsilon_n \end{Bmatrix} = [D_i] \{\epsilon_i\} \quad \text{.. (6)}$$

Here k_{st} denotes shear stiffness and is equal to the shear force sufficient to produce unit shear strain or unit tangential displacement, and k_{nt} is the normal stiffness and is equivalent to the normal stress sufficient to produce unit normal strain or normal differential displacement.

5. MATERIAL PROPERTIES USED FOR INTERFACE

The usage of correct material properties is very important in any analysis. Unfortunately, there is only a very limited test data available for the interface, especially for the interface between masonry and soil. In the absence of actual test data, the properties reported in Ref. 6 have been adopted for the interface analysis. The value of the shear stiffness, k_{st}, has been calculated using the following equation:

$$k_{st} = \left\{ 1 - \frac{R_f \tau}{C_a + \sigma_n \tan\delta'} \right\}^2 k_j \gamma_w \left(\frac{\sigma_n}{p_a}\right)^{n'} \quad \text{.. (7)}$$

where k_{st} is the tangent shear stiffness, τ and σ_n are shear and normal stresses respectively, γ_w is the unit weight of water, C_a is the adhesion at the interface, p_a is the atmospheric pressure, k_j, n' and R_f are constants.

The values of various parameters adopted in the study are as below:

Unit weight of masonry = 2.4 t/m^3, unit weight of soil = 1.6 t/m^3, unit weight of water = 1.0 t/m^3, cohesion = 0.0 t/m^2, adhesion (C_a) = 0.0 t/m^2 interface friction angle (δ) = 33°, stiffness constant (k_j) = 75000, stiffness exponent (n') = 1 and failure fatio (R_f) = 0.87, τ=1.7kg/cm^2, σ_n=2.28kg/cm^2.

The nonlinearity at the interface has been accounted for by considering an equivalent shear stress of 1.7 kg/cm^2 corresponding to yield stress. The corresponding value of k_{st} has been obtained as 7.55×10^4 t/m^3 using equation (7). The value of k_{nt} has been taken as 6.36×10^{12} t/m^3. The stress analysis has been carried out with these stiffness values for case 3(b). The effect of value of k_{st} on time periods, mode shapes and mode participation factors has been studied for Cases 3(a) to 3(e). For this, six different values have been assigned to k_{st} as given in Table 2.

6. RESULTS AND DISCUSSION

6.1 Time Periods

The time periods for the Cases 1 and 2 are tabulated in Table 1 and for Case 3 in Table 2. It is observed that the time periods decrease with the provision of fill, as expected. Case 2(b) has got the least time period of 0.143 sec, the time period of dam alone being 0.219 sec.

Comparing the time periods of Cases 2 and 3, it is seen that the introduction of the interface element has very little effect on the period.

6.2 Effect of Value of Shear Stiffness

The effect of the values of k_{st} on time periods has been studied for Cases 3(a) to 3(e) by considering six values of k_{st} for each case.

Table 2 gives the values of k_{st} and the corresponding values of time periods for six cases. By examining these values, it can be concluded that as the value of k_{st} increases, the time periods decrease though by a very small amount.

6.3 Stresses

Table 3 gives an abstract of maximum principal compressive and tensile stresses due to individual and combined actions of various loads for all cases considered. The stresses obtained for various cases are discussed below:

(i) Case 1

The maximum principal compressive and tensile stresses have been obtained, respectively, as 108.5 t/m^2 and 97.0 t/m^2 due to static loads, and 134.3 t/m^2 and 130.2 t/m^2 due to static plus dynamic loads. The results indicate that the vertical normal stress across any horizontal section does not vary linearly and this fact is more prominent on the lower position of the dam where the width is large.

(ii) Case 2

Due to static loads, the maximum tensile stresses for Cases 2(a) through 2(e) are, respectively, as 31.0, 14.7, 28.3, 39.4 and 52.1 t/m^2. These values are lesser than that without backfill and the percentage reduction in the stresses ranges from 46% to 85% depending upon the backing profile.

Due to static plus dynamic loads, the maximum tensile stresses for the above cases are, respectively, as 117.7, 79.3, 112.1, 107.2 and 87.4 t/m^2.

The percentage reduction in stresses in this case ranges from 10% to 33% as compared to case 1.

(iii) Case 3

In this case the maximum tensile stresses due to static and combined loads are, respectively, as 31.4 and 69.0 t/m^2. The corresponding values without interface elements in Case 2(b) were found as 14.7 and 79.3 t/m^2. Thus the introduction of interface element is to decrease the tensile stress by about 12% for combined loads.

9. CONCLUSIONS

The earthbacking could reduce the tensile stresses in the dam. The percentage reduction varies from 10 to 33% for static plus dynamic loads and from 46 to 85% for static loads depending on the backing profile. Various alternatives should be tried to arrive at the most effective profile.

The effect of introducing interface elements on time periods and stresses has been found to be marginal for the value of k_{st} reported in the paper.

TABLE 1 - TIME PERIODS IN SEC. FOR CASES 1 AND 2

S.N.	CASE	MODE 1	MODE 2	MODE 3
1	1	0.219	0.087	0.058
2	2a	0.153	0.070	0.064
3	2b	0.143	0.071	0.070
4	2c	0.150	0.070	0.064
5	2d	0.159	0.076	0.061
6	2e	0.174	0.084	0.058

TABLE 2 - TIME PERIODS IN SEC. FOR CASE 3 (Ist MODE)

S.N.	Value of k_{st} (t/m^3)	Case 3(a)	Case 3(b)	Case 3(c)	Case 3(d)	Case 3(e)
1.	7.55×10^4	0.155	0.146	0.151	0.159	0.175
2.	5.28×10^4	0.152	0.142	0.149	0.158	0.174
3.	2.50×10^7	0.152	0.142	0.149	0.158	0.174
4.	5.92×10^7	0.152	0.143	0.149	0.159	0.174
5.	8.18×10^7	0.152	0.143	0.149	0.205	0.174
6.	1.08×10^8	0.152	0.142	0.149	0.158	0.174

TABLE 3 - MAXIMUM PRINCIPAL STRESSES (t/m^2)

S.N.	CASE	STATIC LOADING Compressive	STATIC LOADING Tensile	STATIC + DYNAMIC LOADING Compressive	STATIC + DYNAMIC LOADING Tensile
1	1	-108.5	97.0	-134.3	130.2
2	2a	- 97.5	31.0	-129.4	117.7
3	2b	-101.6	14.7	-139.3	79.3
4	2c	- 96.6	28.3	-137.4	112.1
5	2d	- 91.8	39.4	-119.7	107.2
6	2e	- 87.7	52.1	-121.5	87.4
7	3b	-105.7	31.4	-183.1	69.0

REFERENCES

1. Murti, N.G.K., "State of Art on Earth backing of Masonry Dams", Irrigation Power, Journal of C.B.I & P., Oct. 1979, Vol. 36, No. 4 (ICOLD Special).

2. Zienkiewicz, O.C., "The Finite Element Method", 3rd Edition, McGraw-Hill, 1977.

3. IS:6512-1972, "Criteria for Design of Solid Gravity Dams", Second Reprint, April 1981.

4. Menon, K.K.M., "Dynamic Analysis of Composite Gravity Dams", M.E. Dissertation Submitted to University of Roorkee, Roorkee, Oct. 1982.

5. Sharma, H.D., Nayak, G.C. and Maheshwari, J.B., "Generalization of sequential Nonlinear Analysis - A Study of Rockfill Dam with Joint Elements", Numerical Methods in Geomechanics, Vol II, ASCE, 1976.

6. Clough, G.W. and Duncan, J.M., "Finite Element Analysis of Retaining Wall Behaviour", Journal of Soil Mechanics and Foundation Division, ASCE, Vol. SM12, Dec. 1971.

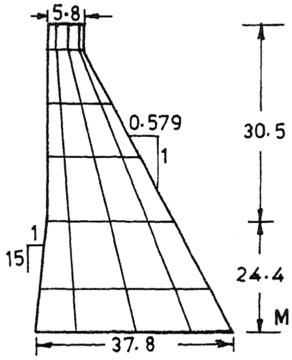

FIG. 1
MASONRY DAM SECTION

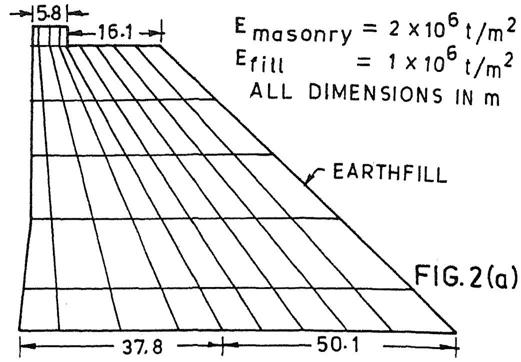

FIG. 2(a)

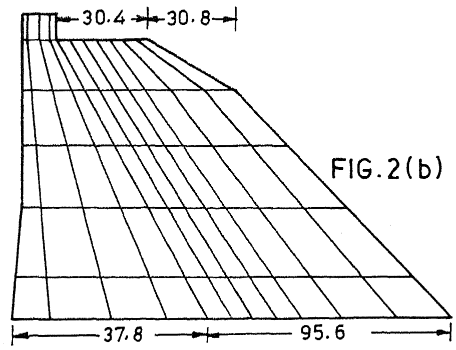

FIG. 2(b)

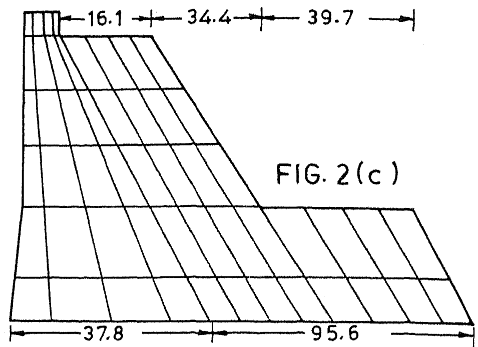

FIG. 2(c)

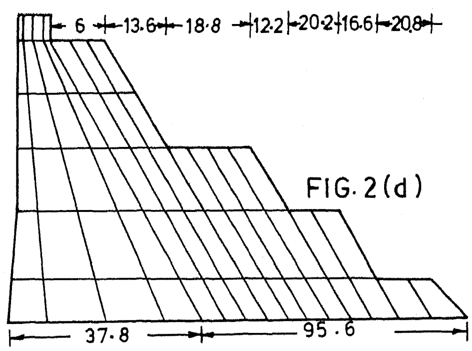

FIG. 2(d)

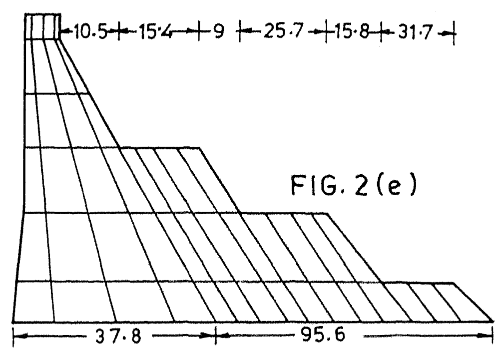

FIG. 2(e)

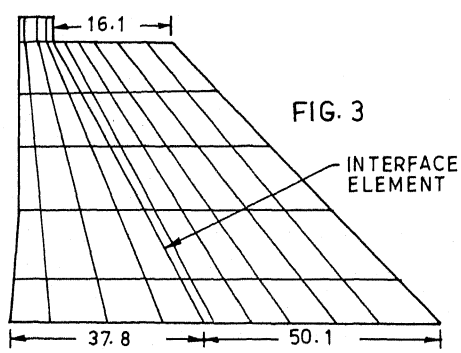

FIG. 3

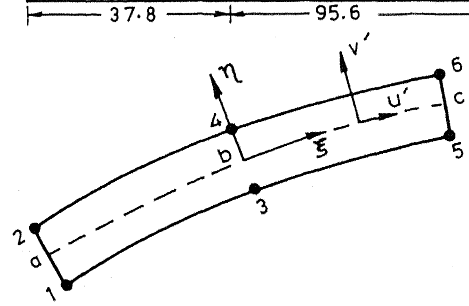

FIG. 4 _ INTERFACE ELEMENT