BASIS OF DESIGN DECISIONS IN STRENGTHENING A DAM SUBJECTED TO EARTHQUAKE DAMAGE-A CASE HISTORY

by

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SYNOPSIS

The Koyna dam which was subjected to a severe earthquake in December, 1967 had to be strengthened in two stages. A temporary stage in which pre-stressing and epoxy grouting of the cracked portion of the non-overflow monolith was resorted to and a permanent stage in which the blocks were backed with mass concrete. The basic considerations on which the decisions for restorative measures were taken by the designers are briefly reviewed in this paper. It is shown that purely analytical approach had several limitations and the decisions had to be essentially based on observations on the existing dam, constructional convenience and codes of practices in different countries.

INTRODUCTION

Design decisions involve anticipation of the probable maximum forces acting on a structure during its useful life and an assessment of the material strength characteristics at that time. These forecasts are based on the available information. In the dynamic design of dams there is a great paucity of accurate information and decision may have to be taken mainly on the basis of similar existing structures. Case histories of important dams subjected to earthquake forces indicating the assumptions made in such designs will not only assist future designers but will also identify the direction in which research effort should be concentrated to fill up the large gaps in the existing knowledge. Koyna dam provides a unique example in which the original design was based on the assumption that the dam was located in a seismically stable area but had later to be strengthened against a strong earthquake. Extensive literature (1-6) is now available on the seismicity of the Koyna region and on the structural response of the Koyna dam to the December 1967 earthquake. Design considerations for arriving at the restorative measures have,however,not received adequate attention. An attempt has been made in this paper to briefly review the important considerations on which the various measures for restoration were adopted.

KOYNA DAM AND THE EARTHQUAKE

The Koyna dam is a rubble concrete gravity dam, 103 meters high at the deepest foundation. Each block is about 16 m wide and the total length of the dam is 854 m. The joints between the blocks are ungrouted. The dam is founded on massive basalt. The upstream elevation and typical overflow and non-overflow sections are shown in Fig.1. A peculiar feature of the overflow section is a kink at the downstream face of the dam where the slope changes from .725:1 to .153:1. This dam was originally planned

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to be constructed in two stages. Later when the first stage construction had already progressed it was decided to raise the dam to its full height and this necessitated thickening of the top of the non-overflow section from the stability considerations (1). The dam was designed with the assumptions that full uplift acted on the upstream face reducing to 2/3 at the galleries and tail-water level at the downstream toe, no tension under normal load and a small tensile stress for earthquake acceleration of .05 g.

On December 11, 1967 an earthquake of magnitude estimated (3) around 6.5 occured giving a severe jolt to the structure. Although due to power failure some of the instruments did not function the accelerograph in block No.1A had worked and gave a rare accelerogram (3). The maximum acceleration recorded in this accelerogram in the horizontal transverse direction was 0.51 g, in the horizontal longitudinal direction 0.66 g and in the vertical direction 0.36 g. The duration of the strong shaking was about 5 secs and the accelerogram showed eighteen crossings per second (8).

TEMPORARY RESTORATIVE MEASURES

The dam is the key structure in the Koyna Hydroelectric Project supplying power to an important industrial region in the country. The project was commissioned in 1962 and had a installed capacity of 540 M.W. As any permanent measure to strengthen the dam would take a period of at least a couple of years, it was necessary to resort to some temporary measures to ensure as much security as possible. It was necessary that the power supply be continued and therefore the reservoir level should be maintained well above the head-race tunnel. Detailed inspection of the dam showed that the spillway section which had an over-hang showed remarkably small distress. There was some wetness along the corners in the galleries at the foundation level and at the intermediate level and a fine crack at the valve chamber where the distance from the upstream was small. The most significant damage observed was in the non-overflow blocks. Most of these blocks showed a crack both at the upstream and at the downstream face near about the level of the kink. In many of the blocks water could be seen seeping through these cracks.

For temporary restorative measures, therefore, the following decisions could be taken on the basis of the above observations and other supporting data :

1. It was not necessary to strengthen the overflow sections which showed little distress.
2. In the non-overflow sections the greatest distress was at the level of the kink where analysis (3,4) showed high dynamic stresses and the shear friction factor was the minimum.

It was decided first to close the crack at the upstream end by epoxy grouting and at the same time pre-stress the top portion of the block by stressed cables as shown in Fig.2. These cables would serve a double function of not only increasing the normal stress and thereby increasing the shear resistance but also enable anchoring the top portion of block to the main body thus preventing large displacements. Initially it was proposed to anchor the cables to the foundation.

However as the time available was too short from the construction angle, it was decided to anchor the cables at an intermediate level in the body of the dam. The further upstream could the cable be located the more effective it was in stressing the upstream face but this was likely to introduce tensile stresses below the anchor zone. After a number of tests, both on micro-concrete models and photoelastic models the optimum distance of three meters from the upstream face was decided upon (7).

PERMANENT RESTORATIVE MEASURES

The pre-stressing by cables was small and could not be a final solution to the problem. For a permanent solution two alternatives were considered in the initial stage. The first one consisted of a backing of the downstream face of the dam by earth or rock fill. This was later, rejected on the ground that the conditions at the interface of the rock and the concrete could not be easily visualised and the behaviour of such a composite dam under earthquake loads will be very difficult to estimate. The solution finally considered was to back the downstream face of the non-overflow section by concrete.

A number of studies using mathematical and physical models of various types were carried out (2,4,5). At the initial design stage a rapid experimental method based on Stadola's static analogue (4) was used for determining the mode shapes and frequencies and later the response spectra method in conjunction with photoelastic method to obtain the stresses. The results are indicated in Fig.3. The final design was based on the Japanese (2) work which involved the determination of the frequency and mode shapes of various blocks by constructing gelatine models and vibrating these on shaking table. The acceleration spectrum obtained from block 1A was used to assess the acceleration $\ddot{G}(\tau)$, at the base of the dam and this data in conjunction with the model data used to determine the response of the dam. The analysis was conducted on a digital computer using the basic equation,

$$\alpha(x,t) = \sum_{i=1}^{2} F_i(x) C_i p_i \int_0^t \ddot{G}(\tau)\, e^{-h_i p_i (t-\tau)} \sin p_i (t-\tau)\, d\tau$$

where $\alpha(x,t)$: acceleration of response vibration
$F(x)$: mode of normal vibration
p : circular frequency of the natural vibration
h : the damping constant
C : the participation coefficient $= \dfrac{\int_0^l F\,dm}{\int_0^l F^2\,dm}$

where dm is the elemental mass of the monolith and l the height. The subscript 'i' denotestthe mode.

The horizontal seismic force which acts on the monolith at any elevation is given by multiplying the response acceleration with equivalent mass. Bending moments and shears were calculated by the traditional methods. When the computed values were compared with the traditional method it was found that the stress condition in the dam could approximately be reproduced by adoption of uniformly distributed transverse coefficient of 0.5. Although the compressive stress during the earthquake was well within the ultimate strength of concrete used (175 to 225 kg/cm^2) the tensile stress exceeded 30 Kg/cm^2 even after taking a damping factor as high as 12%. This was much more than the expected tensile strength. However the dam had infact stood well except for the distress at the level of the kink. Consequently it was decided to adopt the conventional method of design adopting uniform seismic coefficient of 0.15 - 0.20 for the design of the restorative measures. This decision was based on the observations on various dams in Japan as well as an assessment of the nature of damage that had occurred in the Koyna dam. Redesigned section of the dam is shown in Fig.4. For rapidity and ease in construction it was decided to limit the backing on the entire face upto the level indicated, above which backing was in the form of a buttress.

DISCUSSION

During the design of the restorative measures for the Koyna dam it became apparent that the analytical techniques for dynamic analysis and static material properties could not give quantative information for arriving at design decisions. The design ultimately involved the following assumptions :

i) The Koyna dam is not likely to be subjected to any earthquake greater than the one that had occured on 11th December 67.

ii) The observed behaviour of the non-overflow monoliths was entirely satisfactory and therefore no strengthening is necessary.

iii) All analytical and experimental work as well as observations showed that the kink in the dam was not desirable. The downstream slope of the dam should be straight as far as possible with the batter of the order of 0.8 : 1.0.

iv) The standard codes of practice with seismic factor ranging from 0.15 to 0.20 could be used for the design of the restorative measures.

v) Modifications could be made in the downstream backing to enable rapidity and ease in construction and that the effect of longitudinal vibration of buttresses was negligible.

Observations of the response of the Koyna dam and other related works to the earthquake have shown that the dam and other structures have considerable reserve dynamic strength. Factors that contribute to this high reserve are not yet clear. In case of Koyna dam this may be due to the particular characteristics of the accelerogram (9) or to the existence of compressive thermal stresses (8) (Fig.5) or a high damping after initiation of cracks or other unknown factors. The case history of the Koyna dam emphasizes that dynamic analysis by simple and rapid methods (4) must be done for all important dams for a qualitative assessment of the stress distribution and large stress concentrations in the first and second modes should be avoided.

CONCLUSIONS

At the present stage of our knowledge regarding the probability of occurrance of an earthquake and serious lack of understanding of the mechanics of materials under dynamic earthquake loads with varying characteristics, the designer must necessarily base his decisions on the observations on existing structures. More emphasis must be placed on qualitative optimization rather than on quantative evaluation. Restorative measures adopted for Koyna dam were in the final analysis guided by the observed behaviour of the dam under the December 1967 earthquake and on profiles of dams which had withstood earthquakes in other countries. There is an imperative need of having a better understanding of the dynamic behaviour of materials before the refined analytical methods of stress analysis could be effectively utilised in the dynamic designs of these structures.

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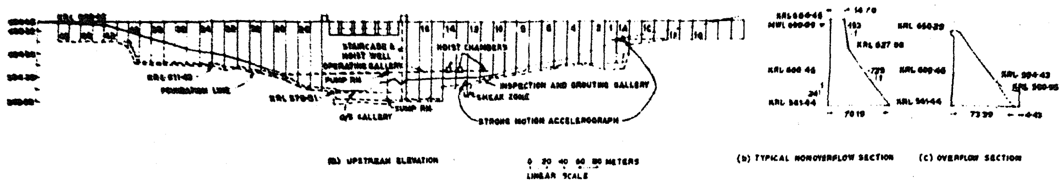

FIG. 1 KOYNA DAM-ELEVATION AND SECTIONS.

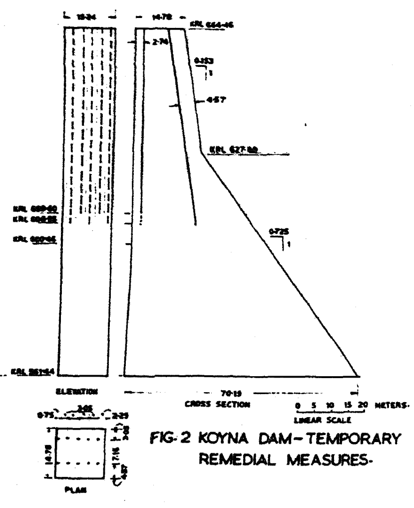

FIG-2 KOYNA DAM-TEMPORARY REMEDIAL MEASURES.

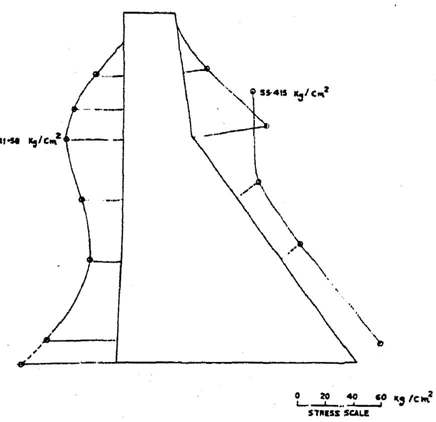

FIG. 3 KOYNA DAM-DYNAMIC PEAK BOUNDARY STRESSES.

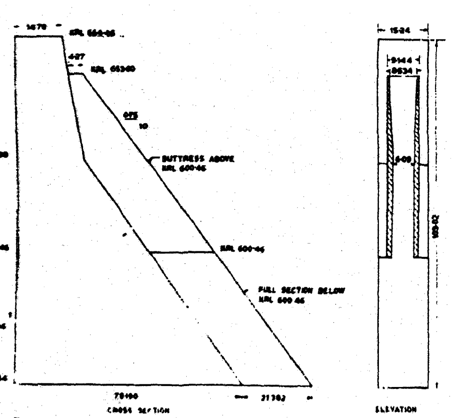

FIG 4 KOYNA DAM-PERMANENT REMEDIAL MEASURES.

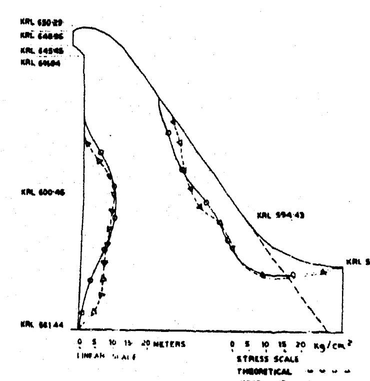

FIG. 5 KOYNA DAM-THERMAL BOUNDARY STRESSES.

DISCUSSION

Henry J. Degenkolb (U.S.A.)

As a practicing engineer who is responsible for the safety of individual structures, the discussor found this paper to be very refreshing. After the engineer applies all of the analytical tools at his command - thorough analysis, review of various theories of failure and performance and making the best estimates of material performance that he can ultimately he has to rely on past experience and observation. The proof of the weakness in the dam was the damage at the critical point - not an analysis. It is fortunate that the damage confirmed the analysis. If there has been a dis-agreement between the two, the performance would have indicated the obviously correct method and the analysis would have been in error.

In the oral presentation, the author seemed to depreciate the practical and non-mathematical reasons for choosing the final repair solution, referring to it as a retrograde solution. He is unduly modest. When a final choice of design, construction, or repair systems is to be chosen especially in an important structure where many lives are at stake, observation and past experience are the primary, most reliable factor influencing that choice. Observation of past performance of structures is the basis by which the adequacy of our analytical solutions is measured. Dr. Pant is to be congratulated for his courage to state the actual basis of the repair decisions to a room full of so many adherents to the exclusively analytical approach.

D.J. Ketkar (India)

1. For permanent restorative measures, what is the objection to use prestressing cables. If cable protection is the worry, there is good number of dams in this region (Walwan, Shirawta, Tanso, Tulsi) stabilised against uplift conditions by prestressed cables. These dams have withstood the 'Koyna' earthquake without any problem. In U.S.A., cables are being used for this purpose even today. Is there any other deficiency ?

2. Considering the problems/unknowns of

   i) bounding old and new concrete

   ii) shear transfer across the concrete backing

   iii) temporary instability during construction i.e. excavating upto foundation rock.

3. Would it not be beneficial at this stage to study the dams already stressed and also monitor their seismic response?

4. Which among these two methods (cables or concrete backing) is better to resist the shear stresses from successive earthquakes ?

5. Will there be any significant difference in strengthening a concrete structure vis-a-vis a masonry structure.

Author's Closure

Not received.