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ANALYSIS OF THE EARTHQUAKE PERFORMANCE OF KOYNA DAM

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INTRODUCTION

The Koyna earthquake of December 11, 1967 is an event of considerable engineering significance especially because it was centered in the vicinity of Koyna Dam. Koyna Dam is a straight gravity structure made of rubble concrete except for a 6 ft. thick zone of conventional concrete at the upstream face of all monoliths. It is about 2800 ft. long and 338 feet high above the deepest foundation, constructed in 50 ft. wide monoliths, and has a 300 ft. long overflow portion. Typical sections are shown in Fig. 1. Considerations and

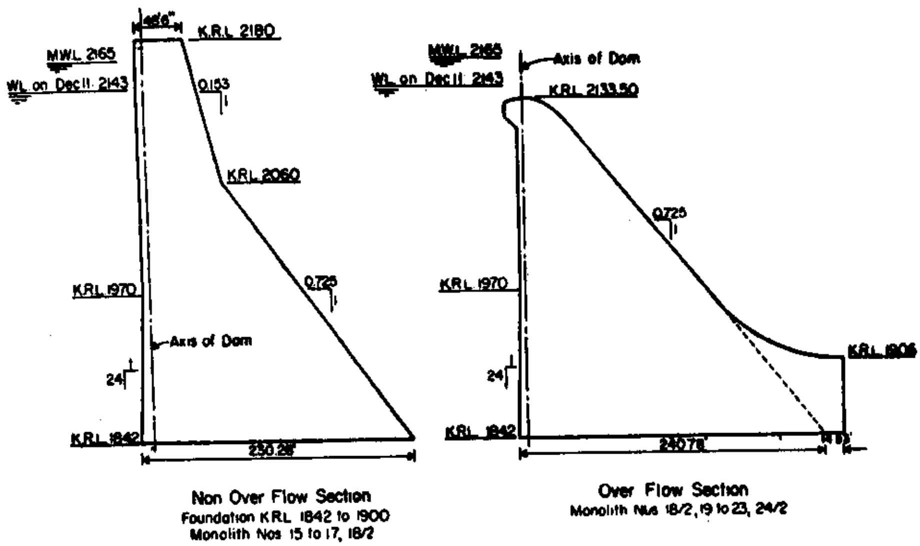

Fig. 1 Koyna Dam—Sections

criteria that had been employed in designing Koyna Dam were similar to those in use in many parts of the world (6, 7, 9, 12). The Koyna section is not typical of gravity dams, however, and this is demonstrated in Fig. 2. The Koyna earthquake was recorded on a strong motion accelerograph located in a monolith near the right bank. It appears reasonable to neglect the distortion of the ground motion due to the dymamic characteristics of this monolith, and consider the accelerograms as representing the ground motion at the site (8). The peak accelerations for the recorded ground motion were 63% g for the longitudinal component, 49% g for the transverse component, and 34% g for the vertical component, and the duration of strong shaking was about 6 seconds (11). The most important structural damage to the dam were horizontal cracks on either the upstream or the downstream face or both faces of a number of monoliths. The principal cracking was in the taller non-overflow monoliths on both sides of the spillway portion around KRL 2060

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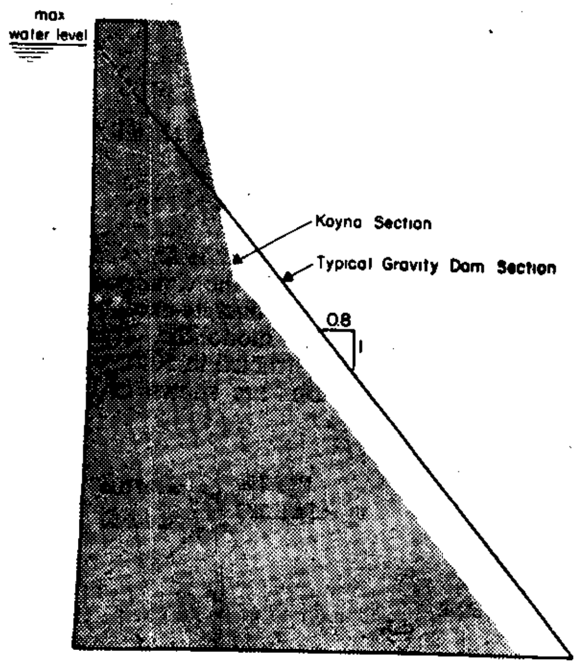


Fig. 2 Koyna and Typical Gravity Dam Section

which is the level at which the slope of the downstream face changes abruptly; the overflow monoliths were undamaged (1,10). The damage was serious enough to require lowering the reservoir for inspection and repairs and to require permanent strengthening. The strengthening of Koyna Dam has been designed and is under construction. The section of overflow monoliths is increased over the entire width from the base up to KRL 1970 and above this level is a buttress of width varying between 20 and 30 ft. (Fig. 3), whereas the overflow monoliths are not being strengthened.

Considerable work has been done during the past decade towards developing techniques for analysis of gravity dams subjected to earthquakes. It is of interest to evaluate these techniques in light of the behaviour of Koyna Dam during the Koyna earthquake. Whether these techniques would have enabled us to predict the damage to the dam during ground shaking of intensity and general frequencies characteristics similar to those recorded during the earthquake, is therefore studied herein. Next, the effectiveness of the strengthening designed for the dam is approximately evaluated. Intuitively, the original Koyna section would appear to be more vulnerable to earthquake damage than a typical gravity section. The response of a more conventional dam section to the Koyna ground motion is therefore analyzed to provide a basis for comparison. This work is based on the report of more complete investigation submitted to the U.S. Department of the Army (5).

ANALYTICAL TECHNIQUES

The length of Koyna Dam is about eight times its height and about eleven times its width, and the vertical joints between monoliths were not grouted which resulted in the various monoliths vibrating somewhat independently during the earthquake (1, 10). In

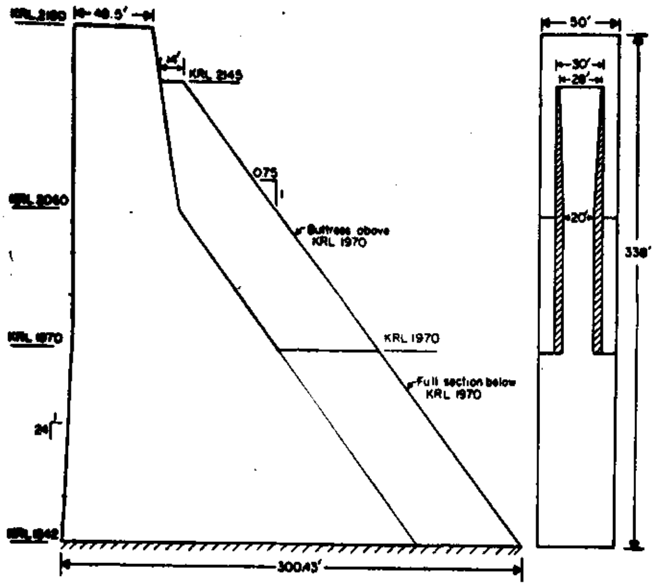


Fig. 3 Strengthened Section of Monolith 17 of Koyna Dam

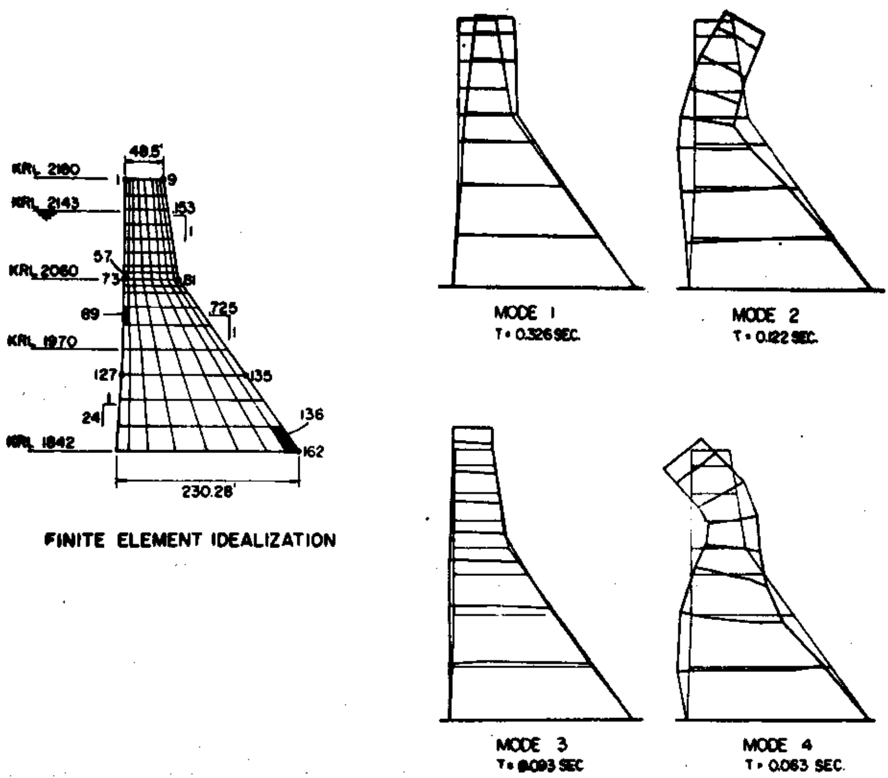


Fig. 4 Natural Periods and Mode Shapes of Non-overflow Monolith of Koyna Dam

analyzing the dam, it therefore appears reasonable to simplify the problem to a two-dimensional vibration of a monolith due to transverse and vertical components of earthquake ground motion. A linear analysis is adequate for the purposes of this investigation. If dynamic interaction between dam and reservoir and between dam and foundation are excluded, the problem can be analyzed most effectively by the finite element method utilizing a recent computer program (2, 3). This method can conveniently handle arbitrary structural geometry and non homogeneities. The computer program is based on the mode superposition method. This method is generally advantageous in case of earthquake excitations as the more important structural response is usually contained in the first few mode shapes.

A good estimate of the influence of dynamic interaction between dam and reservoir on response of dams can be obtained by an available analysis technique (4). Elasticity and inertia of the foundation influence dynamic stresses in dams, but this appears to be a relatively less important factor for Koyna Dam and the base is therefore considered rigid.

ANALYSIS OF KOYNA DAM

Non-Overflow Monoliths

The finite element idealization used in analyzing a typical non-overflow monolith (Fig. 1) is shown in Fig. 4, consisting of 136 quadrilaeral elements and 162 nodal points, which allowing for the constraints at the base provides 306 degrees of freedom. Although the strength of the four different concrete mixes used in the dam is significantly different, their density, modulus of elasticity, and Poisson's ratio are similar (13), and for purposes of a finite element analysis the section is therefore considered to be homogeneous with modulus of elasticity$=4.5\times10^6$ psi unit weight$=165$ pcf, and Poisson's ratio$=0.20$. On the basis of limited experimental results of damping capacities of concrete gravity dams obtained in Japan and recently for Pine Flat Dam, California, it appears reasonable to select 5 percent as the damping ratio for the lower few modes of Koyna Dam.

This finite element system is analyzed by the computer program mentioned earlier. The computer results represent the combined effect of the initial static loads and the earthquake ground motion, and include the complete time-history of the horizontal and vertical components of displacements and accelerations of all nodal points and of the three components of stress in all finite elements. The complete results are too voluminous to be included here and only an extremely small part, which is considered to be the most significant, is presented.

The distribution of initial stresses is shown in Fig. 5, and as provided for by the designers the section is entirely in compression. The first four modes of vibration were included in the mode superposition computations. The periods and shapes of vibration of these modes are shown in Fig. 4. The variation of stresses in two selected elements with time is presented in Fig. 6. The larger Peaks are seen to occur for a duration of about 3 secs. The distribution of critical stresses over the cross section is presented in Fig. 7. Tensile stresses exceeding 500 psi develop at the upstream face between KRL 2040 and 2080. Although the area near the downstream face over which the tension exceeds 700 psi is of comparable size, there is a concentration of stresses near KRL 2060 where the downstream face changes slope abruptly with tension exceeding 1000 psi over a small area. From the computer results, the maximum tension near the heel of the section was about 500 psi and the largest compressive stress in the section exceeded 1250 psi.

The results obtained from an analysis included dynamic interaction between dam and reservoir are presented in Fig. 8. In this analysis, the reservoir level was taken as KRL 2143, the level at the time of the earthquake. The time history of displacement at the crest

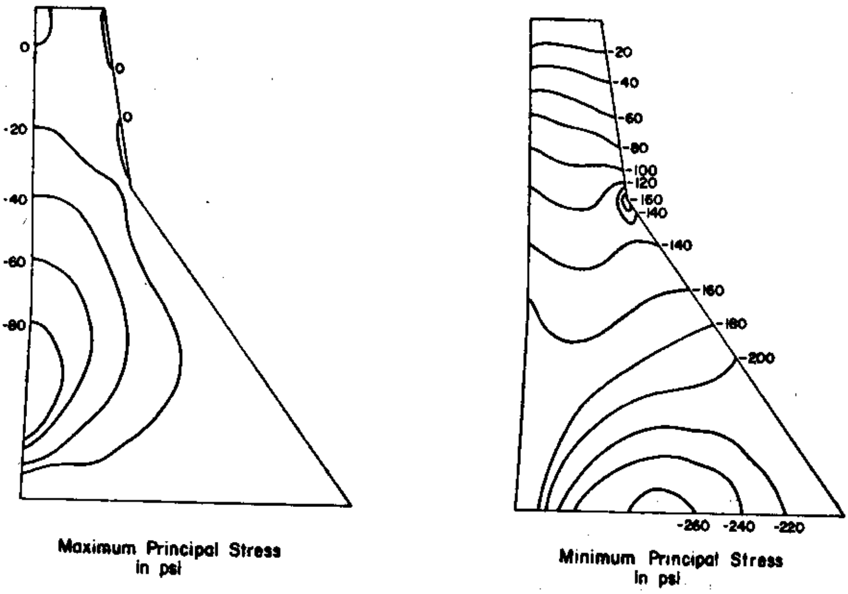


Fig. 5 Static Stresses in Non-overflow Monoliths of Koyna Dam

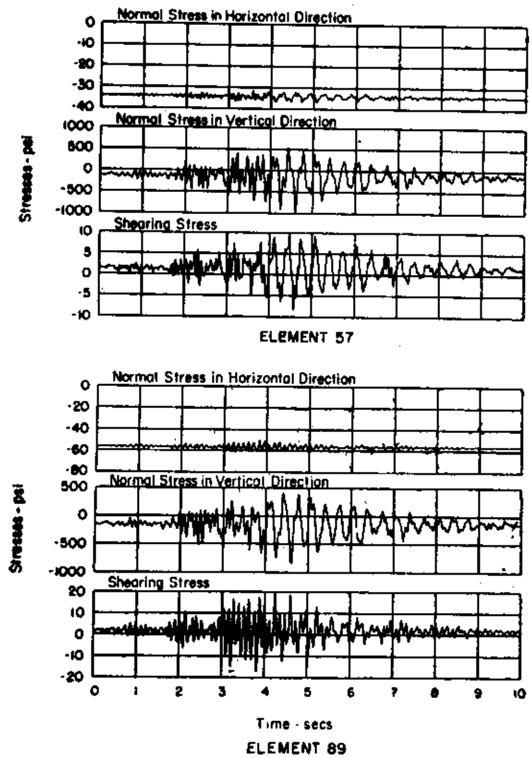


Fig. 6 Stress Responses of Koyna Dam to Transverse and Vertical Components of Koyna Earthquake

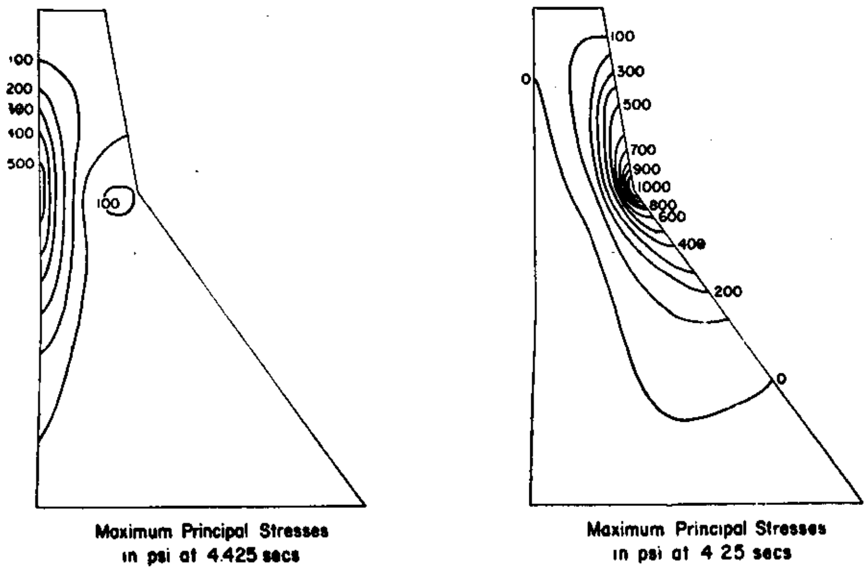


Fig. 7 Critical Stresses in Non-overflow Monoliths of Koyna Dam Due to Transverse and Vertical Components of Koyna Earthquake

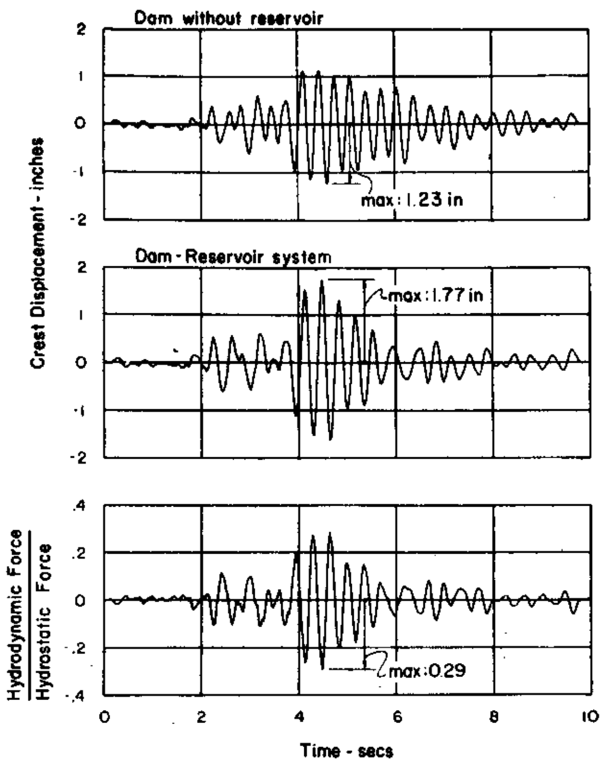
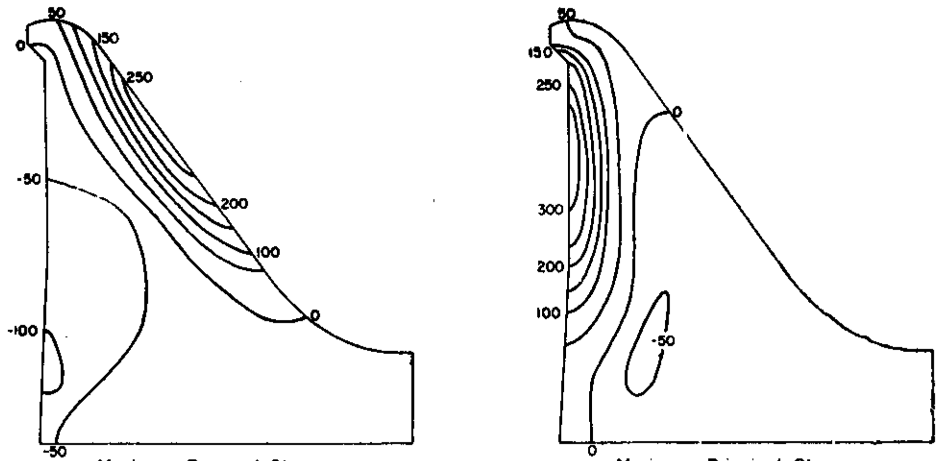


Fig. 8 Response of Koyna Dam-Reservoir System to Koyna Earthquake

of the dam without the reservoir is shown, and it has a maximum value of 1.23 in. When the hydrodynamic pressures and the dynamic interaction between dam and reservoir are included, the time-history of the crest displacement has a longer dominant period reflecting the lengthening of the vibration period of the dam due to the reservoir. The peak value of the crest displacement is 1.77 in. which is about 45 percent larger than the value when the reservoir was excluded. The corresponding peak tensile stresses are of the order of 700 psi between KRL 2040 and KRL 2080 near the upstream face, and 1400 psi at KRL 2060 near the downstream face.

Overflow Monoliths

A similar finite element analysis of a typical overflow monolith (Fig. 1) was performed The most important results of this analysis are presented in Fig. 9, revealing that the maximum tension in this case is considerably smaller. The maximum tensile stress at the



Maximum Principal Stresses in psi at 2.90 secs

Maximum Principal Stresses in psi at 3.825 secs

Fig. 9 Critical Stresses in Overflow Monoliths of Koyna Dam Due to Koyna Earthquake

upstream face is about 300 psi in contrast to about 500 psi in case of the non-overflow monoliths. There is no stress concentration near the downstream face of the overflow section and the maximum stress is only about 250 psi compared to about 1000 psi for the non-overflow section. The increase in these stresses due to hydrodynamic pressures and dam-reservoir interaction has not been computed for this case. If this increase is roughly similar to that in case of non-overflow monoliths, maximum tension would be about 450 psi on the upstream face and 350 psi on the downstream face.

COMPUTED STRESSES AND OBSERVED CRACKING

The pertinent data concerning three of the four different concrete mixes used in Koyna Dam are listed in the table below; only a small of volume of Mix No. 1 was used in the lowest parts of the dam and it is therefore excluded (13). The compressive strength values are based on published data, and the tensile strength is assumed to be about 10 percent of the compressive strength.

Mix. No.	*Parts of the Dam*	*Compressive Strength (psi)*	*Tensile Strength (psi)*
2	up to KRL 1990	4100	410
3	KRL 1900–2160	3500	350
4	above KRL 2160	2900	290

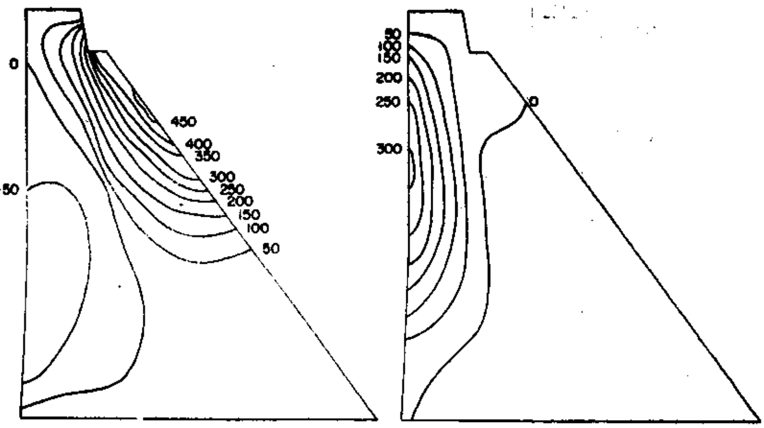
The results of linearly elastic response analyses of the dam presented and discussed earlier indicate that the tensile stresses in the taller non-overflow monoliths exceed the tensile strength of Mix No. 3 from KRL 1900 to 2130 near the faces, by a factor of upto four, over a duration of about 3 secs. Cracking is not likely to occur over the entire zone over which stresses exceed the tensile strength because their spreading would be curtailed by the dissipation of energy in the earliest cracking. This is generally consistent with the cracks observed in monoliths 15 through 18. Analysis of other non-overflow monoliths would lead to a similar correlation between anticipated and observed cracks. Tensile stresses in the overflow monoliths of Koyna Dam computed earlier are of magnitude similar to the tensile strength of Mix No. 3. Consequently, little or no cracking would be expected to occur in the overflow monoliths which is consistent with observed damage.

It appears to be a common practice in India to use concrete mixes of different strengths in different parts of a gravity dam; the highest strength concrete is used in the lower parts of the dam and the strength gradually decreases at higher elevations. This practice would appear to be logical if the stresses increase with distance below the top of the dam. The loadings considered in conventional design procedures for gravity dams include self weight, hydrostatic pressures, uplift forces, and additional static forces to represent earthquake effects. All of these design loads increase with distance below the top, and the resulting stresses are smallest near the top of the dam and increase at lower elevations (Fig. 5). The above mentioned practice of decreasing the concrete strength at higher elevations is probably based on such considerations. However, a dynamic analysis of stresses in gravity dams due to earthquakes demonstrates that the stresses do not decrease at higher elevations; on the contrary, the most critical stresses occur in the upper part of the dam. The concept of decreasing the concrete strength at higher elevations therefore does not appear to be a sound one; on the contrary, from the earthquake point of view relatively higher strength concrete would be perferable in the upper part of the dam.

EVALUATION OF STRENGTHENED SECTION

The width of a strengthened non-overflow monolith of Koyna Dam (Fig, 3) is not constant over the section. A three-dimensional analysis would be required for determining the earthquake response of such a monolith. However, the objective of this study was restricted to an approximate evalution of the earthquake performance of the strengthened dam. It was therefore deemed adequate to idealize the monolith by an equivalent section of uniform width with modified material properties, and to perform a two-dimensional plane stress finite element analysis. Furthermore, the dynamic interaction between the dam and the reservoir is ignored and the foundation is assumed to be rigid.

The periods of vibration of the first four modes of the strengthened monolith computed by such an analysis are 0.24, 0.10, 0.091 and 0.057 sec which are smaller than those for the original monolith The reduction is about 25 percent in the fundamental period, but much smaller for the higher modes. The response of the strengthened dam to Koyna ground motion is analyzed by the computer program mentioned earlier. The distributions of critical stresses over the cross-section of the strengthened dam are presented in Fig. 10. Because the strengthened monolith is simplified to a two-dimensional plane stress system in the analysis, these results represent, at best, the average stresses over the width of the monolith. The largest tension at the downstream face is approximately 450 psi which is about half the corresponding value for the original non-overflow section. The largest tension occurs in the upper one-third height of the buttress and there is no local stress concentration associated with it. The maximum value of tension on the upstream face is 300 psi which is about 60 percent of the corresponding value for the original section. It is apparent that significantly smaller stresses develop in the strengthened monolith, resulting in a significant improvement in its earthquake behavior.



Maximum Principal Stresses in psi at 2.925 secs

Maximum Principal Stresses in psi at 3.675 secs

Fig. 10 Critical Stresses in Strengthened Koyna Dam

ANALYSIS OF TYPICAL GRAVITY DAM SECTIONS

Analyses presented above demonstrate that large tensile stresses must have developed in Koyna Dam during the Koyna earthquake thus resulting in significant cracking of the dam. It is not obvious whether the apparently high stresses develop because the cross-section of Koyna Dam is not typical of concrete gravity dams, and it is therefore of interest to analyze the response of a typical gravity dam section. The dam chosen to be representative of typical concrete gravity dams is Pine Flat Dam on King's River near Fresno, California, (Fig. 11). Although many other dams have similar sections and they are typical of concrete gravity dams, Pine Flat Dam is chosen because its height (400 feet) is roughly comparable to the height of Koyna Dam and it would therefore have similar periods of vibration.

The dynamic interaction between dam and reservoir is ignored, and the foundation is assumed to be rigid in this analysis. The properties selected for mass concrete in this dam are : modulus of elasticity $=5\times10^6$ psi, unit weight $=155$ pcf, and Poisson's ratio $=0.20$. The computer program mentioned earlier is used to analyze the response of this monolith subjected to the Koyna ground motion. The periods of the first four modes of vibration are 0.256, 0.125, 0.092 and 0.072 sec. Damping is assumed to be 5 percent of critical in each of these four modes included in the response analysis. The distribution of maximum principal stress over the cross section at two selected instants of time is presented in Fig. 12. The stresses at the downstream face attain a maximum tensile value of approximately 800 psi, tensile stresses exceeding 600 psi occur at the upstream face in the upper part of the dam, and according to the computer results the maximum compressive stress in the section is about 950 psi. In comparing these results with the response of Konya Dam, it is apparent that the maximum tensile stresses developed at the downstream face are approximately 20 percent smaller, whereas those at the upstream face are approximately 20 percent larger; the maximum compressive stresses at the upstream face are of similar magnitude and those at the downstream face are significantly smaller. Tensile stresses exceeding 300 psi develop at the heel of Pine Flat Dam whereas a 500 psi tension occurs in a corresponding zone of Koyna Dam.

Although there are differences in the response of the two dams to Koyna earthquake, it is apparent that comparably large tensile stresses develop in both the dams. Although

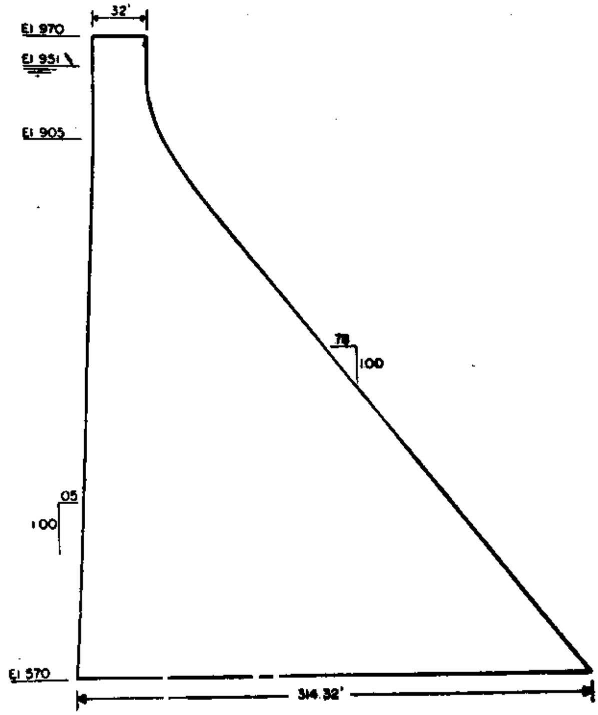


Fig. 11 Pine Flat Dam

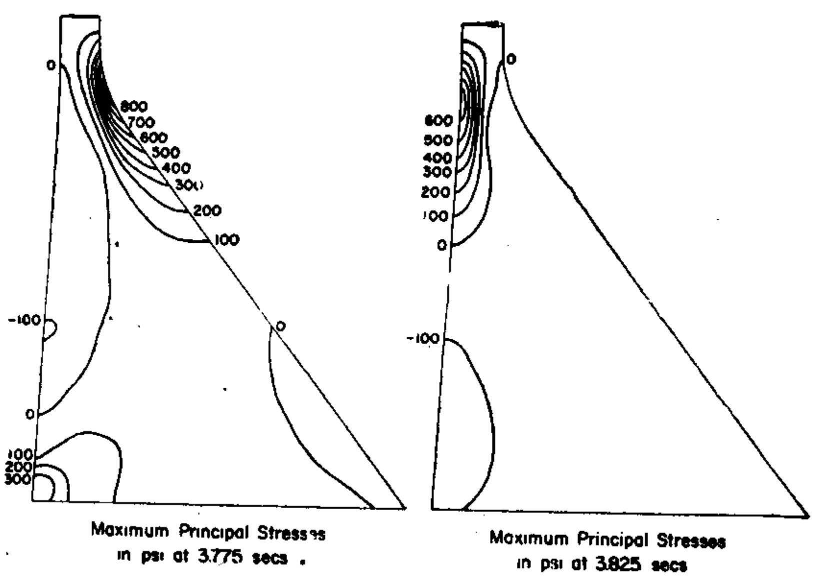


Fig. 12 Critical Stresses in Pine Flat Dam Due to Koyna Earthquake

the cross-section of Koyna Dam is not typical of gravity dams and at first sight would appear to be significantly more vulnerable, it is apparent from the results presented above that typical concrete gravity dams, for example Pine Flat Dam, of height and concrete strength similar to that at Koyna Dam would be expected to suffer comparable damage during an earthquake similar to the Koyna earthquake.

CONCLUSIONS

1. The state of the art in earthquake analysis of gravity dams has advanced to the point that a reasonable prediction of the damage likely to be caused to a given dam by a given ground motion can be made.

2. On the basis of approximate analysis, the strengthening designed for Koyna Dam appears to result in a significant improvement in the earthquake behaviour. A more complete analysis of a strengthened monolith including three-dimensional aspects and the question of cumulative damage in the original portion of the monolith would be necessary before any definite conclusion concerning its earthquake resistance can be made.

3. Although the cross section of Koyna Dam is not typical of gravity dams and at first sight would appear to be more vulnerable, analysis demonstrate that typical concrete gravity dams of height and concrete strength similar to that at Koyna Dam are likely to suffer comparable damage during an earthquake similar to the Koyna earthquake.

4. Dynamic analysis demonstrate that the more critical tensile stresses occur in the upper parts of gravity dams. The practice of decreasing the concrete strength at higher elevations in dams, which was employed at Koyna Dam, is therefore not a sound one; on the contrary from the earthquake point of view relatively higher concrete strength is required in the upper parts of dams. It would be useful to provide relatively higher strength concrete for a few feet thickness at the upstream and downstream face in the upper parts of gravity dams, because the tensile stresses are largest at the faces.

5. The damage to Koyna Dam by the Koyna earthquake shows that concrete gravity dams are not as immune to earthquake as has been commonly believed. Significant tensile stesses develop near the faces and at the heel. The present practice for design of gravity dams needs to be modified to recognize that such stresses can occur and to provide for their consequences.

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