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SEISMIC RESPONSE OF A RETROFITTED CONCRETE BUTTRESS DAM

M. GHAEMIAN

Department of Civil Engineering
McMaster University
Hamilton, Ontario, Canada L8S 4L7

ABSTRACT

The Sefid-rud concrete buttress dam suffered severe structural damage during the Manjil earthquake (Iran) in 1990. The most serious damage to the dam consisted of horizontal cracks that ran the length of the dam at about 14 m below the crest at both upstream and downstream faces of the dam. The tallest monolith was severely damaged as compared with other monoliths. Following the earthquake, retrofitting work of the dam was initiated. Repairs were made by grouting the cracks and installing prestressed anchors in the cracked regions.

The objective of this study is to evaluate the linear response of the Sefid-rud dam to the Manjil earthquake and the behaviour of the retrofitted dam when subjected to the maximum design basis earthquake (DBE) and the maximum credible earthquake (MCE). A dynamic analysis of the most damaged monolith is conducted using the finite element method. The longitudinal and vertical components of the Abbar record, back-calculated at the dam site, are taken as the maximum credible earthquake at the region. The design basis earthquake has been estimated as 0.57 of the maximium credible earthquake.

Theoretical prediction of the overstressed elements in the finite element model of the critical monolith is shown to correspond to the observed crack location. The threshold of damage to the dam from the Abbar recorded ground motion is estimated to be at a peak ground acceleration of 0.617 g. The retrofitted Sefid-rud concrete buttress dam is found to have adequate strength to withstand the design basis earthquake (DBE). However, when subjected to the maximum credible earthquake (MCE) damage to the dam is expected.

KEYWORDS

Anchor, buttress dam, dynamic analysis, grouting, monolith, post-tensioning, retrofitting

INTRODUCTION

The June 20, 1990 Manjil-Rudbar (Iran) earthquake of magnitude $M_s = 7.7$ was extremely destructive. The earthquake caused a reported total of 40,000 deaths, left more than 500,000 homeless in the densely populated area of the western Alborz mountain, south-west of the Caspian sea in Iran.

The Sefid-rud dam was seriously damaged during the Manjil earthquake. The dam was built during the period of 1958-1962 on the Sefid-rud river near the city of Manjil in Gilan province. It is a non-reinforced concrete buttress dam that is 106 m high with crest length of 425 m. The dam consists of 30 monoliths (4 gravity and 26 buttress). The monoliths are 14 m wide with buttress web thickness of 5 m and footing thickness of 6 m. The upstream slope of the dam face (horizontal / vertical) is 0.4 and the downstream slope is 0.6. The buttresses are separated by vertical joints. A typical monolith of the Sefid-rud dam is shown in fig. 1.

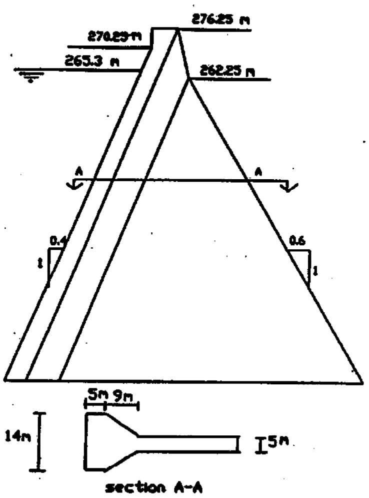

Fig. 1 Typical monolith of the Sefid-rud dam

The epicentre of the earthquake was located at about 10 km from the dam site near the city of Rudbar. At the time of the earthquake, the reservoir was almost full at an elevation of 265.3 m . There were no strong motion recording instruments on or in the vicinity of the dam at the time of the main event. However, records of the earthquake were made at Abbar and Ghazvin which are located at about 47 km and 80 km from the epicentre, respectively.

Following the earthquake, the most serious damage to the dam consisted of horizontal cracks that appeared in the upper parts of the monoliths, especially the tallest one at the centre of the dam where the crack ran almost the whole length of the monolith at elevations 262.25 and 260.25 on the upstream and downstream, respectively. A major crack ran almost the whole length of the dam at about 14 m below the crest. Site investigations following the earthquake indicated that the dam monoliths moved independently with permanent differential displacements of up to 50 mm. These large relative displacements damaged the seals in the contraction joints and caused water leakage from the joints. The high acceleration of the ground motion in the longitudinal direction of the dam caused pounding between the monoliths.

GROUND MOTION

Ground accelerations of the Manjil earthquake were recorded at fifteen stations at various distances from the epicentre. There are no record of the ground acceleration near the epicentre of the earthquake. The nearest station to the Sefid-rud dam site was located at Abbar, about 40 km from the dam site. Ghazvin is another station, about 80 km from the dam site. From the geological point of view, the recording instrument at Abbar is located on rock while the Ghazvin record is made on hard alluvium deposits. For this reason, the Abbar record was selected to represent the ground motion at the dam site which has similar rock ground conditions.

The ground motion at the dam site was estimated by back-calculating the measured ground motion at Abbar station. Five attenuation formulas were used to determine the peak ground acceleration (PGA) of ground motions at various distances from the epicentre. These formulas are Naumoski, Milne and Davenport, Donovan, McGuire and Campbel (Naumoski, 1984 and Trifuance et al. 1975). The PGA was estimated at epicentral distances of 10 km from the Sefid-rud dam site and 47 km from the Abbar station (Ghaemian, 1993). The ratios of the PGA at these two location were determined. These ratios were subsequently averaged to find a relative increase of peak ground acceleration between the two epicentral distances. The Abbar ground acceleration record was scaled by the average ratio of the five attenuation formulas (=1.818) to obtain a time history representing the ground motion at the Sefid-rud dam site. The ground accelerations back-calculated from Abbar records for the longitudinal and vertical components of the Abbar record are shown in fig. 2.

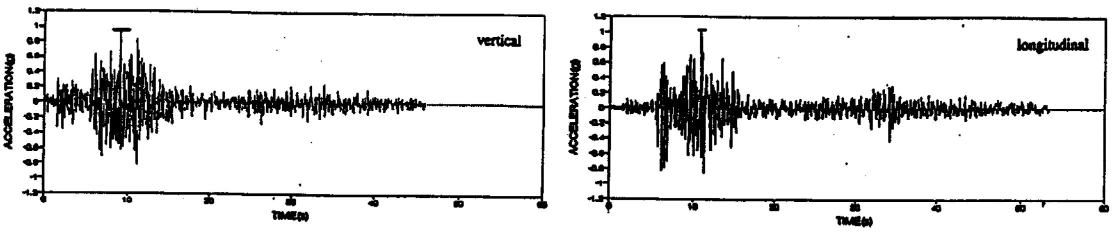

Fig. 2. Back-calculated ground accelerations of Abbar at Sefid-rud dam site

DYNAMIC ANALYSIS OF THE SEFID-RUD DAM

The dynamic analysis of the Sefid-rud dam was conducted using a modified version of EAGD-84, a computer program developed by Fenves and Chopra (Fenves et al. 1984). This computer code was written for the purpose of evaluating the response of concrete gravity dams, including the effect of dam-reservoir-foundation rock interaction. The EAGD-84 program is based on the assumption of a vertical upstream face of the dam. In order to determine the hydrodynamic pressure for the case of inclined dam face, a simplified approach based on experimental data was adopted. For a dam of inclined upstream face, the hydrodynamic pressure distribution can be obtained by multiplying a factor to the pressure obtained for the case of a dam with vertical face and compressible water (Ghaemian, 1993).

The compressive strength of the dam concrete was taken equal to 16.5 MPa with unit weight of 22.9 KN/m^3 and elasticity modulus of 20 Gpa (Ahmadi et al., 1992). The apparent dynamic tensile strength of the Sefid-rud dam using Raphael's equation (Raphael, 1984) is estimated to be f_r = 4.19 MPa. In this dynamic analysis, the modulus of elasticity for the dam concrete is increased by 33% above the uniaxial static value and the Poisson's ratio is taken equal to 0.2. Idealization of the foundation rock underlying the dam as homogeneous and isotropic requires the use of average properties. The average unit weight of the foundation rock layers is 25.28 KN/m^3, the elasticity modulus is 9.94 Gpa and the poisson's ratio is 0.2. The modulus of elasticity of the foundation rock was increased by 25% to account for the uncertainty about the foundation layers. The Wave reflection coefficient of the reservoir bottom material was taken as 0.67. This value is selected to account for the heavy sediment in the Sefid-rud dam reservoir. A constant hysteric damping factor of $\eta_s = 0.1$, which corresponds to a 5% viscous damping ratio ξ (i.e. $\xi = \eta_s/2$) is assumed for all vibration modes of the dam. The velocity of sound in the water is taken as 1,440 m/s. The finite element discretization of the dam tallest monolith is shown in fig. 3.

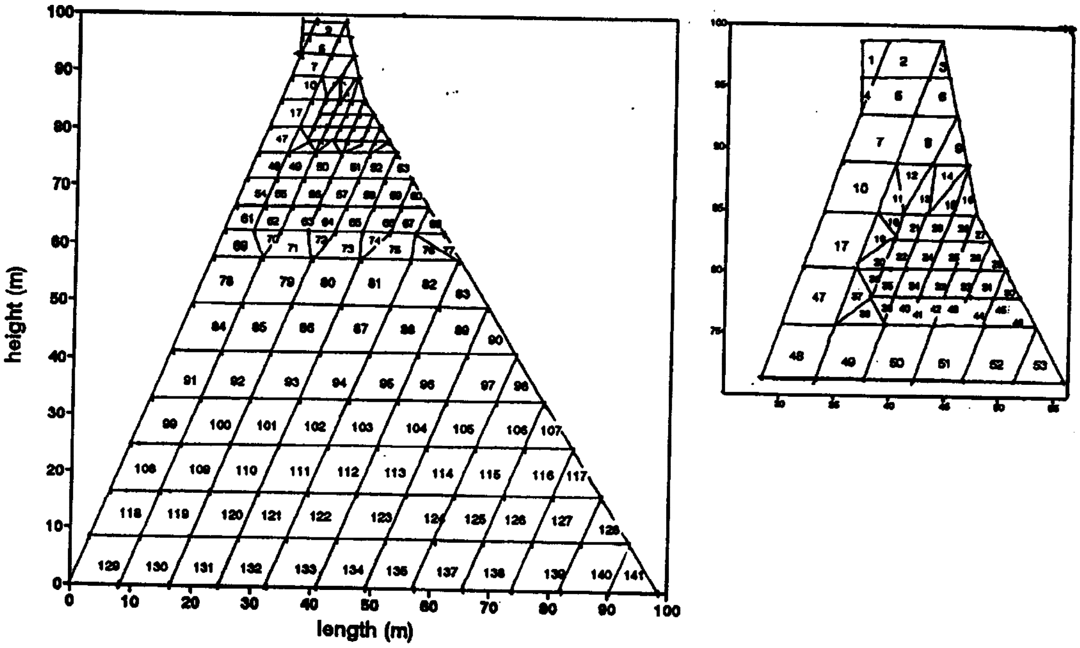

Fig. 3. Finite element mesh of the tallest monolith of the Sefid-rud dam

The maximum tensile and compressive stresses resulting from the dynamic analysis of the tallest monolith are shown in fig. 4. In the analysis, the scaled longitudinal and vertical components of the Abbar records are simultaneously applied to the dam. The tensile stress contours show that at the upper part of the downstream face of the dam, stresses are much higher than the actual tensile strength of 4.19 MPa. Elements 27, 28, 29, 30, 46, 53, 60, 68, 77 and 76 have stresses that exceed the tensile strength of concrete. The principal tensile stress time history of the overstressed elements showed that At t=9.6 seconds, all overstressed elements reached their maximum stresses.

From the analysis of the maximum dynamic stress, element number 27 is determined to be the first element to experience maximum tensile stress and is expected to experience the first crack. The observed location of the crack following the earthquake at the downstream face of the dam (up stream view) of the tallest monolith is at elevation 262.25 m. This elevation coincides with the location of element number 27 in the dynamic analysis.

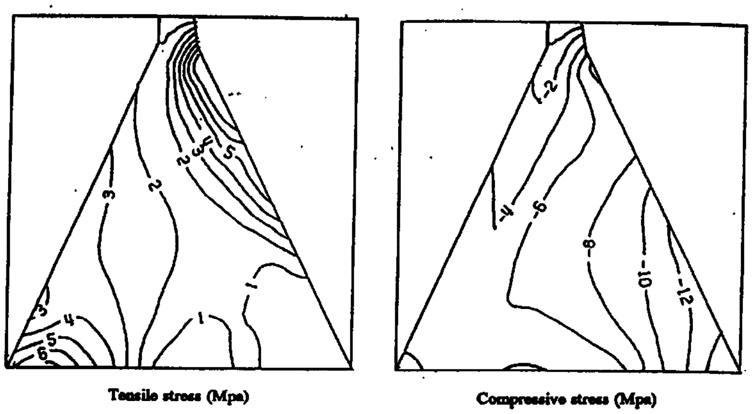

Fig. 4. Maximum compressive and tensile stress contours in tallest monolith of the Sefid-rud dam

The threshold of damage to the dam is defined as the start of initial tension cracking. The threshold of damage can be correlated to the PGA level of the input ground motion that causes cracking of the Sefid-rud dam which is the PGA level associated with the maximum tensile stress of 4.19 MPa. The maximum tensile stress of 4.19 MPa is obtained when the dam is subjected to the Abbar records scaled to PGA = 0.617 g. This PGA represents the threshold of damage to the Sefid-rud dam from that record.

RETROFITTING SYSTEM

Rehabilitation work was done in two stages. The first stage included grouting of the cracks with epoxy resin. The second stage involved the installation of prestressed anchors. Using access from the roadway on top of the dam, 23 to 25 boreholes of 46 mm diameter were drilled in each buttress. Some of the holes were vertical and some were inclined up to 27 degrees. About 1.5 to 3 tons of resin was injected in each buttress to fill cracks due to the earthquake, detachment of concrete working joints, concrete porosity, honey combing and micro-cracking of the concrete mass due to shrinkage and thermal effects. The overall treated surface was estimated to be 6000 m^2. The drilling of over 667 boreholes covered a total length of 15,000m.

Upon completion of the resin grouting, the second stage of repair involved reinforcing the dam body to upgrade its load resistance. In this stage, 12 boreholes of 254 mm diameter and about 42 m length were drilled in each buttress with different slopes (up to 22 degrees) in the upstream and downstream directions of the dam. A total of 234 boreholes for a cumulative length of 9450 m were drilled for the installation of the prestressed anchors. After drilling the boreholes, anchor blocks were cut in the size of 1.5 m deep and 1.3 m wide. The anchors were installed in the borehole with their bond length extended in an area free from cracks. The anchors are designated as VSL type with unbonded free length that is fully grouted. The anchor consists of 54-7 wire steel strands with their free length greased and sheathed with a grout hose in the middle of the strands bundle. The stress in the anchor is transferred to the concrete through a bond length at the bottom and to the dam crest at top by the anchor block. Grouting was performed after installing the anchors. A total of 738 tons of cement and 36,600 kg of additive were injected.

A typical post-tensioning anchor used in the retrofitting of the Sefid-rud dam has the following main components: anchor head, free length, bond length and reinforced concrete block. The anchor head is capable of transferring the tensile load from the tendons to the reinforced concrete block. The free length is the variable distance between the bearing plate embedded in the anchor block and the top of the bond length. The bond length is the designed length necessary to transfer the full capacity of the anchor to the surrounding concrete mass. The reinforced concrete block is located within the dam crest. It includes an embedded stand pipe and a protective cap. Its function is to transfer the anchor force to the body of the dam.

The 7-wire steel strands were selected in accordance with ASTM A416-85. A typical anchor that was installed at the Sefid-rud dam has a minimum working load of 8,400 KN and a guaranteed minimum breaking load of 14,000 KN with an average length of 42 m. Twelve anchors were installed at various angles in the tallest monolith. Post-tensioning of the twelve anchors was done in two phases, 6 anchors in each phase. Long term monitoring of the prestress load was provided for only two anchors. A prestress loss of 3.1% was considered in the calculation of the effective force of the anchors.

The force in the anchor is modelled as a static force acting on the nearest nodes in the direction of the anchor. The location of the concentrated forces due to prestressing of the monolith are shown in fig. 5. The force transmitted to the dam concrete at the lower part of the anchors is assumed to be concentrated at two points. The top point is located at 70% of the bonded length, while the bottom point is located at the end of the anchor. Eighty and twenty percent of the anchor force are applied at the top point and the bottom point of the bonded length, respectively. The anchor force on the dam crest was considered as a single concentrated force.

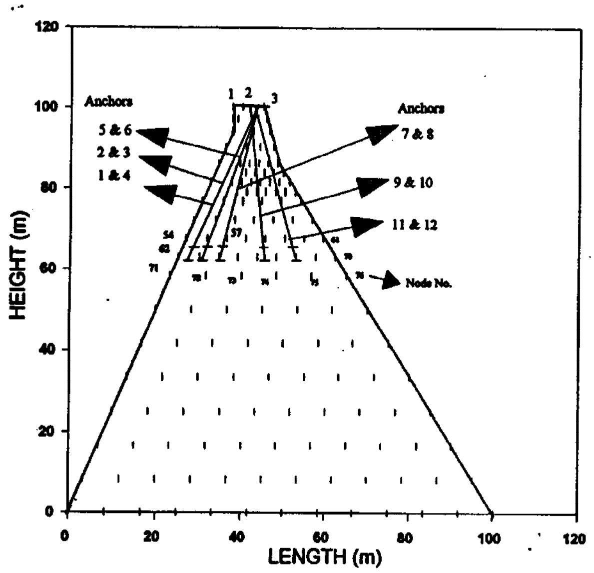

Fig. 5 Location of the anchor concentrated forces in the finite element model

DYNAMIC ANALYSIS OF THE REPAIRED MONOLITH

The Manjil-Rudbar earthquake of 1990 was adopted as the maximum credible earthquake (MCE) in that region. The design basis earthquake (DBE) has been estimated at 0.57 of the maximum credible earthquake. The back-calculated record of the Abbar station at the dam site was used as representative of the MCE and DBE.

The evaluation of the rehabilitation work for the Sefid-rud dam is based on the calculated maximum tensile and compressive stresses in the dam body. The computed stresses should remain below the actual strength of the dam concrete (probable strength). The results of the analysis for the simultaneous longitudinal and vertical components of the DBE ground motion are shown in figure 6. The tensile and compressive stresses are below the actual tensile and compressive strength of the dam concrete.

Also, the dam was analyzed when subjected to the longitudinal and vertical components of the MCE ground motion simultaneously. Results of the analysis for the maximum tensile and compressive stresses in the tallest monolith of the Sefid-rud dam are shown in figure 7.

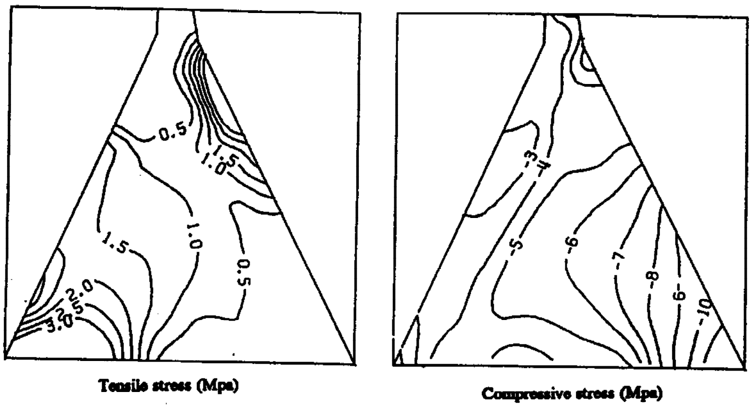

Fig. 6. Maximum compressive and tensile stress contours of the retrofitted monolith under the DBE

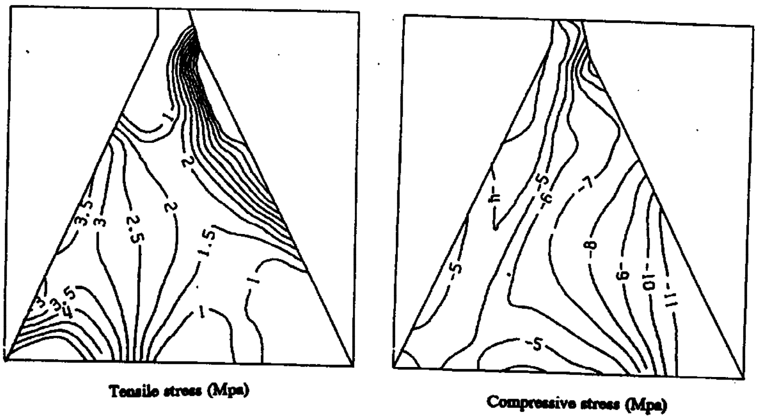

Fig. 7. Maximum compressive and tensile stress contours of the retrofitted monolith under the MCE

DISCUSSIONS

The evaluation of the retrofitted system was only based on the stress analysis of the dam tallest monolith. The retrofitting caused a reduction in the maximum tensile stress at the upper part of the dam due to the existence of the anchor compression force.

The results show that under the Design Basis Earthquake (DBE) of the Abbar back-calculated record, the maximum tensile and compressive stresses are below the tensile and compressive strength of the dam concrete. For the case of the Maximum Credible Earthquake (MCE), the tensile stress in some elements is greater than the tensile strength of the dam concrete. It can be concluded that the amount of stress reduction due to anchor action was not enough to bring the maximum tensile stress below the allowable limit. At elements near the upper part of the downstream face, stresses exceeded the actual strength of concrete. It was found that the stress reduction at the upper part of the dam is more than that at the lower part. This is an indication that a more efficient design of the retrofitting scheme could have been selected. Sloping the anchors toward the downstream face of the dam will increase the compression on the elements at the downstream face of the dam. Also, the anchors in the monolith were concentrated more at the upstream face of the dam which is not an effective position to decrease the tensile stresses in the upper part of the downstream face, the region with the highest tensile stress. In addition, there may be areas that are already weakened by the earthquake and the effect of a future earthquake may be difficult to predict.

CONCLUSIONS

A study was conducted to evaluate the dynamic response of the Sefid-rud dam when subjected to the 1990 Manjil earthquake. Based on the dynamic analysis performed here, it was found that the model of the tallest monolith was able to predict a tension crack at the same location as the one observed after the earthquake took place. There may be a crack at the heel of the dam but there is no information regarding the cracking of the monoliths near the foundation. Also, the retrofitted Sefid-rud concrete buttress dam is found to be capable of withstanding the design basis earthquake (DBE) which is assumed to be 0.57 of the Manjil earthquake. However, for the MCE which is taken equal to the 1990 manjil earthquake, the dam is found unsafe. The safety criteria considered is such that maximum tensile and compressive stresses of the dam concrete remain below the actual tensile and compressive strengths of concrete.

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