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Behaviour of Gilan dams during Iran earthquake of June 1990

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ABSTRACT: In this paper the behaviour of 3 dams, situated within a 50km radius of the epicentre of the Iranian earthquake of 20 June 1990 is discussed. The largest and most important of these dams is the 106m high aseismic-designed Sefid-rud buttressed dam. Situated in the epicentral area of the quake, it survived the 0.65+ g ground accelerations with some cracking in the main body and buttresses and in particular in the intersection of the crown and the main body. Two other dams in the area, Sangar and Tarik, both diversion dams also survived the earthquake. Failure in six of the thirteen steel gates of the Sangar dam and spalling of concrete due to pounding of the bridge deck against the piers in the Tarik dam were the main damage in these two dams. Two important seismic design considerations absent in these dams include; appropriate seismic joints and the safety of secondary systems and equipments.

1 INTRODUCTION

The Manjil earthquake of June 1990 devastated a large, densely populated rural and urban area of the north-west provinces of Gilan and Zanjan. This region is a well-watered agricultural and industrial area. As a result a number of large engineered structures such as dams, ground-based storage tanks, water-towers, silos, bridges and industrial plants were affected by the earthquake. The three dams namely, Sefid-rud dam Tarik dam and Sangar dam were all situated within a distance of 50km from the epicentre and in areas where ground shaking had caused severe damage to residential and non-residential buildings. In the following the earthquake performance of these dams and their unique or common shortcomings are discussed.

2 SEFID-RUD DAM

Sefid-rud dam, one of the largest dams in the Middle East, is situated approximately 2km north-west of the town of Manjil where it collects the waters of the Ghezelozan and Sefid-rud rivers. It is 106m high in the middle section and 425m long at the crest. It's buttressed construction consists of 26 monoliths, each 14m long. There are four end-monoliths of non-buttressed gravity type. The slope of the dam on the downstream face is 1 in 0.6 and on the upstream side 1 in 0.4. It has a vertical crown section 14m high and 10.5m wide. The reservoir was almost full at the time of the main event, being just below the crown section.

The behaviour of this aseismic-designed dam, near the epicentre of the earthquake revealed many information, much attainable from the analytical solutions but some only from the observations, about the actual response of such dams to earthquake loading. The Sefid-rud dam was designed in the 1950's. Construction began in 1958 and was completed by 1967. Because of the importance of the structure, as the main source of electricity generation for the region and the history of seismic activities in the area, the seismic safety was an important consideration in the design of this dam. In those days however the seismic design of structures was carried out using the equivalent static approach. The seismic factor adopted for the design of this dam was 0.25 (Moinfar and Naderzadeh 1990). This is evidently a high factor compared to similar designs of the day, which reflects a conservative approach in design.

The location of Sefid-rud dam in relation to the epicentre of the quake is not yet clearly established. The epicentre was initially located at only 300m north-west of the dam on the line of a ground rupture running almost parallel to the face of the dam. It is now believed that the epicentre was about 10km north of the dam near the town of Rudbar on the line of the main, 80km long, fault associated with the quake and that the ground rupture near the Sefid-rud dam is a tributary of that main fault. What is clear however is that Sefid-rud dam was situated in the area of strongest ground shaking (estimated X on MKS). Unfortunately, there were no seismographs on or in the vicinity of the dam at the time of the main event to record the level of ground accelerations. The nearest accelerograph at Abbar close to the line of the seismic fault but some 40km from the dam recorded maximum acceleration of 0.65 g in the east-west direction and 0.2 g in the north-south direction. From these and from the near total destruction of the nearby town of Manjil it can be assumed that the accelerations suffered by the dam in the said directions were in excess of these values. Also all the indications are that, similar to ground shaking in Abbar, the strongest component of the quake (0.65+) was in the east-west direction, almost parallel to the face of the dam and the line of the main fault (Maheri 1990) and that in the all-important transverse direction (N-S) maximum ground shaking at the site could not have been more than 0.25 g. Such mode of ground shaking could clearly be deduced from the failure modes of the buildings in the nearby town of Manjil and more readily from the response of the small stone masonry guard blocks on the east side of the dam which were all thrown off their footings in a direction parallel to the face of the dam by the strongest component of the quake.

Sefid-rud dam survived the earthquake with relatively little damage, most repairable and have since been repaired (Ahmadi 1991). In the following the main structural and non-structural damage sustained are discussed.

2.1 Structural Damage

The structural damages visible in the main body of the dam can be summarized as follows;

i) Horizontal Cracks: The main structural failure in the dam consists of a long, horizontal crack in the upper parts of the monoliths. This crack, running almost the whole length of the dam about 1.6m below the crown, was less evident in the end monoliths but could easily be seen in the middle monoliths where the oozing of water out of the crack had left some clear marks on the downstream face. In this section the crack had evidently crossed the whole thickness of the dam but appearing at two different levels on the upstream and downstream faces, roughly 4m apart (Fig. 1). The crack appears to have been of the out-

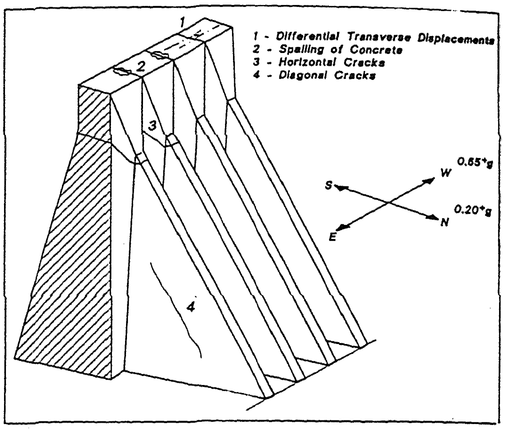

Fig. 1 Sefid-rud dam. Upstraem view.

of-plane bending type caused by the N-S component of the quake. The effects of compressive and tensile failures at the cracked section could be seen in the form of spalling of concrete. Also after the formation of the crack in the middle monoliths, the periodic compressive loading around the crack on the downstream had actually driven a 4m high wedge of concrete, some few millimeters out of position. results of analysis carried out by Ahmadi [3] later confirmed this mode of failure and the location of the crack. These analyses indicated that the initial failure was of tensile type on the downstream face of the dam. Based on the analytical findings, this section of the dam has since been repaired using pre-tensioned cables (Ahmadi 1991).

ii) Diagonal Cracks: As a result of high earthquake-induced shear stresses in central monoliths some diagonal cracks were also developed in the supporting buttresses. These shear failures did not appear significant enough to require remedial measures.

iii) Pounding Action of Monoliths: Evidence of pounding of the monoliths against each other could be seen on the crest in the form of spalling of concrete at

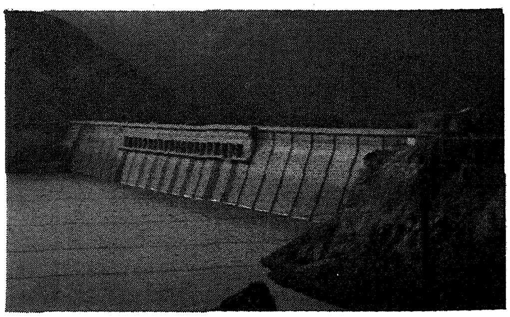

Fig. 2 Structural failures of Sefid-rud dam.

Fig. 3 Sefid-rud dam. Partial view of downstream.

the interfaces (Fig. 4). The pounding was caused by the high-acceleration, longitudinal (E-W) component of the earthquake. The extent of damage due to pounding at the interface of the monoliths could not be investigated. The type and size of seismic joints, if they were at all a consideration in design of the dam, were inappropriate to mitigate the damaging effects of pounding.

iv) Differential Displacements of the Monoliths: As was mentioned earlier, this dam consists of 30 monoliths separated from each other by construction and expansion joints. Under the ground motion certain transverse (N-S direction) differential movements of the monoliths occurred. These differential movements, although not visually noticeable, were measured at a maximum of 50mm from the original alignments of the benchmarks on the crest of the dam. It could not be ascertained whether the measurements relate to the movements of monoliths as a whole or only parts of the monoliths such as the crown at some concrete lift joints. Considering the overall dimensions of the dam, such relative movements could be considered insignificant.

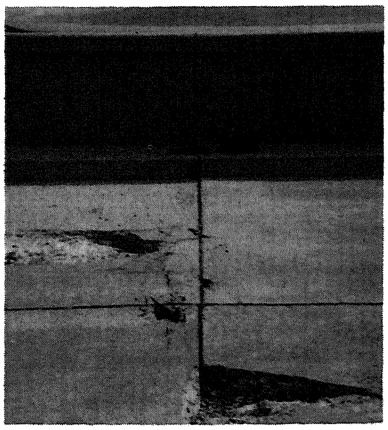

Fig. 4 Sefid-rud dam. Pounding failure

2.2 Non Structural damage

The earthquake also caused some non-structural damage to the associated structures of the dam. These include;

i) Destruction of the guard post at the east side and the guard house in the west side (Fig. 5) of the dam under the impact of the falling rocks. The reinforced concrete guard house was completely destroyed causing at least one fatality.

ii) Flexural failure and collapse of the central sections of the reinforced concrete parapet wall of the north side (Fig. 6). The failure of this long, unsupported wall was in the form of vertical cracks together with horizontal cracks at the base due to bending effects.

iii) Subsidence of the fill adjacent to the concrete section of the dam, particularly on the west side, caused by the compaction of the loose fill under ground vibration.

iv) Overturning failure of the majority of concrete and stone masonry guard blocks, most of which were thrown off their footing

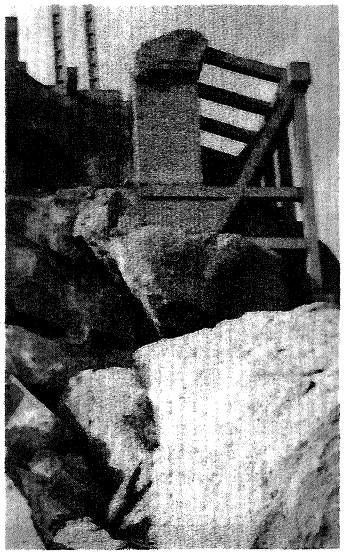

Fig. 5 Destruction of the gaurd house under falling rocks.

by up to two meters in the west direction.

2.3 Damage to Installations

Under the earthquake loading two of the six turbines of the dam reportedly went out of action. The reinforced concrete structures supporting the turbines sustained various degrees of damage. The steel gate of one of the inlets was severely deformed. This gate which was shut at the time of the quake must have experienced large impulsive hydrodynamic pressures. Other damages to the installations include; collapse of the un-reinforced masonry infills of the control room causing severe damage to this room and some damage in the switch yard where the massive transformators were lifted off the rails and were thrown some 25cm in the horizontal direction (Eshghi 1991). Lifting off the rails of these transformators indicate high vertical accelerations of the quake at the site.

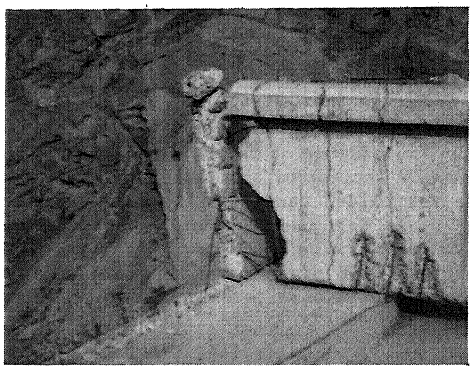

Fig. 6 Sefid-rud dam. Failure in the parapet wall of the north side.

3 TARIK DIVERSION DAM

This dam situated an estimated 25km north of the epicentre experienced much less ground accelerations than the Sefid-rud dam. This can be deduced from the isoseismal map of the area and the behaviour of the dam itself. Tarik dam is 350m long of which the concrete section measures 230m and runs, similar to the Sefid-rud dam, in an east-west direction (Fig. 7). It consists of 10 concrete piers each 3m thick and 23m high. The piers measure 20m at the crest and 54m at the base. The dam directs 32m^3 of water per second through a channel 17km long to the agricultural lands around Fuman. The flow of water through the dam is controlled by semi-cylindrical steel gates each 15m long and about 8m high. The movement of these gates is controlled individually by an automatic pulley system. The bridge deck of the dam runs on the north side and level with top of the piers. The deck rests on columns supported by the piers (Fig. 8).

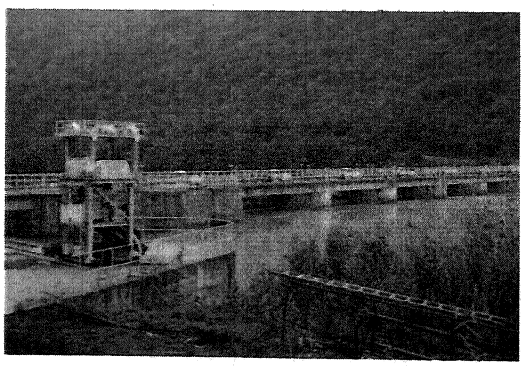

Fig. 7 Tarik diversion dam. Upstram view.

There were no visible damage to the concrete section of the dam under direct earthquake loading. However there were local cracking and spalling of concrete at the top of almost all the piers in close contact with the bridge deck (Fig. 9). The failures were evidently caused by pounding action of the relatively flexible bridge deck against the rigid piers under the N-S component of the earthquake. The distance between the bridge deck and the concrete piers was not sufficient to accommodate the relative flexible responses of the two almost independent sections of the dam. Another revealing form of failure in the dam could be seen in the extreme east side of the dam where the stone support had completely crushed. This indicates high ground accelerations in the E-W direction. A situation similar to the site of the Sefid-rud river. None of the failures mentioned above were serious and the overall behaviour of this dam during the earthquake could be considered favourable. The steel gates of the dam joining to the concrete piers via rotating steel arms also stayed in

Fig. 8 Tarik dam. Details of the piers and the bridge deck.

place without damage. At the time of the main event, two of these gates were open allowing the water through. This must have somewhat reduced the high levels of hydrodynamic forces on the steel gates during the movements of the dam against the mass of reservoir.

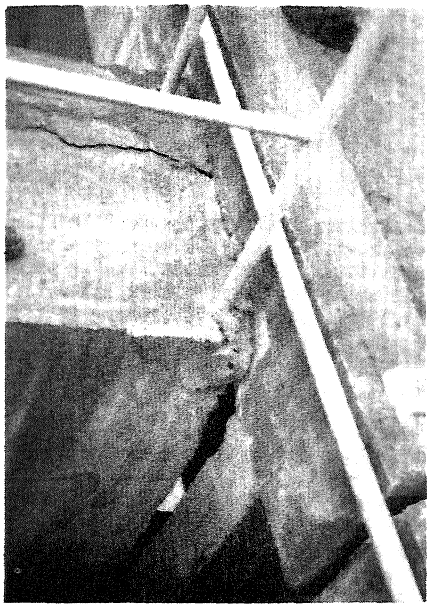

Fig. 9 Tarik dam. Pounding failure in the concrete piers.

4 SANGAR DIVERSION DAM

Further afield from the epicentre, some 60 km north of the Sefid-rud dam, another diversion dam suffered some damage during the earthquake. This dam which diverts the water through two, east and west channels to areas respectively 19km and 25km away, is similar in design to the Tarik dam. It consists of 13 steel gates each with a discharge capacity of $400m^3$ per second. The gate movements are however controlled by counter balancing large concrete blocks in such a way that when the gates are shut (down) these blocks are in a raised position. During the earthquake the ground motion caused dislocation of some of the controlling cables off the pulleys. This resulted in the sudden lowering of the concrete blocks and therefore raising of the steel gates. In total six gates were opened in this way. Under subsequent cycles of ground shaking two of the raised gates were reportedly thrown off their supports and were found about 200m downstream. The moving arms and couplings of the other 4 gates also suffered heavy damage. The seismic behaviour of the Sangar dam clearly indicates the unsuitability of this kind of design for earthquake loading, as the cantilever action of the heavy concrete blocks or the steel gates in a raised position makes them very susceptible to failure. Apart from the failure of the gates the earthquake cause no apparent structural damage to the dam itself. However the diversion channels suffered heavily and particularly the west channel, cover of which over a distance of 1.8km collapsed.

5 CONCLUSIONS

1- All he indications point to the fact that the strongest component of the Manjil earthquake happened to be parallel to the face of the three dams discussed. Particularly in the case of the Sefid-rud dam it should be noted that although the dam was situated in the area of severest ground accelerations (max. 0.7g), it did not experience more than maximum 0.25g acceleration in its critical direction (i.e. north-south). Hence the overall good performance of the dam should be considered in the light of this fact.

2- In the cases of the Sefid-rud dam and the Tarik dam, some damage occurred as a result of the pounding of different sections of the dams against each other. The importance of seismic joints, particularly when separating sections with different dynamic properties are therefore evident.

3- Much damage was caused to the secondary systems, elements and installations of the dams. Inadequate and inappropriate connections between these systems and their supporting structure were responsible for these damages. This illustrates the point that as far as the economy is concerned, in the seismic design of lifeline structures such as dams the safety of secondary structures, systems and equipment is as important as the integrity of the main structure.

REFERENCES

Ahmadi, M.T. 1991. Dynamic response of Sefid-rud buttress dam during the Manjil-Roudbar earthquake. Proceedings of the first international conference on earthquake engineering and seismology, Tehran, Iran, vol. III 331-344.

Eshghi, S. 1991. The Manjil-Rudbar earthquake of Khordad 31, 1369, Iran. Technical report 1, Publication No. 70-1, (in Farsi)

Maheri, M.R. 1990. The engineering aspects of the Manjil, Iran earthquake of 20 June 1990, A field report by EEFIT, W.S. Atkins Engineering Sciences, U.K., Publication

No. G8630/90/01.
Moinfar, A.A., Naderzadeh, A. 1990. An immediate and preliminary report on the Manjil, Iran earthquake of 20 June 1990. Building and housing research centre, Publication No. 119, (in Farsi).