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# Karameh Earth Dam, A Challenging Project

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SYNOPSIS: The under construction Karameh dam is situated in the Dead Sea Rift (32.00°N, 35.50°E), the boundary between the Arabian and African-Sinai plates. The primary seismic source contributing to the hazard at the Karameh dam site is the active Jordan Valley fault which extends from the Dead Sea to the Sea of Galilee (from 30.90°N to 32.93°N at a longitude of 35.50°E), with an expected maximum earthquake magnitude of 7.8, and passes under the right abutment of the dam body. This paper presents the analysis of the dam under the earthquake loading. This includes the stability analysis of dam embankment under seismic loading, the expected displacement of the dam body, and the liquefaction potential and its associated displacement. The results of analysis indicate that the dam can resist an earthquake of magnitude 7.8 without catastrophic failure, and the fault rupture will not exceed 4.0 m. Also, liquefaction may occur in the foundation layers, which is expected to result in a crest settlement of 4.4 m.

## INTRODUCTION

Dams are very important earth, since they store large quantities of water to be used for irrigation, power generation, recreation, etc. Embankment dams made up of a compacted impervious clayey core protected by compacted rockfill material on both sides are common in Jordan, since they are easier to construct and lower in cost compared to gravity dams. Besides, embankment dams proved to perform well in earthquake-prone region.

Earthquake engineers give special attention to the seismic stability of structures due to the fact that great property damage and human losses will result if such structures collapse in densely populated and developed areas. The potential loss of lives and property damage certainly will occur with the failure of dams, and specially if this takes place in heavily populated areas. Therefore, it is essential to pay great attention to the seismic stability of earth dams.

This paper deals with the seismic stability analysis of Karameh dam, which is to be constructed on an earthquake prone site. Due to the uniqueness of geotechnical conditions at the dam site, the effect of bedrock motions on the ground surface motions will be further studied so that the effect of the local soil conditions at the damsite can be incorporated in the study.

## SEISMICITY AT THE DAM SITE

Jordan lies in the area of the Jordan - Dead Sea Rift which extends from the Arabian Shield in the south to the highly movable Zagros Mountain range in the north. Geological and magnetic feature points to a relative plate movement along the rift boundary. (Kovach et al. 1990).

Information about the past earthquakes is used as a basis for seismic hazard estimates. Husein et al. (1994) collected valuable data about historical and the 20th century instrumentally recorded earthquakes for Jordan and vicinity. Table (1) shows the historical and instrumentally recorded earthquakes in and around the dam site. This table indicates that the site of the dam has been affected by major earthquakes of magnitudes greater than 6.0 in the past.

Based on detailed studies by many researchers (Ben-Menahim 1981; and Tapponnier 1992 (JVA-Review Meeting), it has been found that the rate of the slip average value of about 1 cm/year along the Jordan Valley Fault (JVF) and the rate of uplift is about 4 mm/year. Clearly, this is a very significant strike-slip fault.

Faults can be classified into six types according to their activities, starting from 1 to 6, where 1 represents the most active type (Cluff et al. 1982). A fault with the slip rate $\geq$ 10 mm/year, slip per event $\geq$ 1 m, rupture length $\geq$ 100 km, magnitude $M_s \geq 7.5$ and recurrence interval

≥ 1000 years is classified as Class 1A. The Dead Sea Fault is classified as Class 1A fault.

Table 1: Historical and instrumentally recorded earthquakes in the dam site

| Year | Latitude in° | Longitude in° | Surface Wave Magnitude ($M_s$) | Local Magnitude ($M_l$) | Epicentral Intensity ($I_o$) |
|---|---|---|---|---|---|
| 33 | 32.00 | 35.50 | 7.4 | 7.0 | 9.0 |
| 634 | 32.00 | 35.50 | 6.85 | 6.6 | 9.0 |
| 748 | 32.00 | 35.50 | 7.8 | 7.3 | 11.0 |
| 1034 | 32.00 | 35.50 | 6.7 | 6.5 | 10.5 |
| 1160 | 32.00 | 35.50 | 6.1 | 6.1 | 8.0 |
| 1546 | 32.00 | 35.50 | 7.4 | 7.0 | 10.5 |
| 1834 | 32.20 | 35.50 | 6.5 | 6.35 | 9.0 |
| 1903 | 32.10 | 35.50 | 5.25 | 5.50 | 8.0 |
| 1927 | 32.20 | 35.50 | 6.50 | 6.30 | 9.0 |

The method proposed by Cornell (1968) is used in this study to estimate the Peak Ground Acceleration (PGA) at bedrock in which the attenuation equation developed by Esteva 1974 is used herein. The computer program FRISK (McGuire, 1978) using faults as earthquake sources is used in this study. For more detailed information about the methodology, see (McGuire, 1978) According to ICOLD (International Commission on Large Dams, 1989), the operating basis earthquake (OBE) should have a 50% probability of non-exceedance in 100 years lifetime of structures. Seismic hazard maps based on Cornell approach was prepared and shown in Figure (1). For Karameh Dam this represents a return period of 145 years, and a design acceleration of 0.12g. The equivalent 90% confidence level result in a design acceleration of 0.24g.

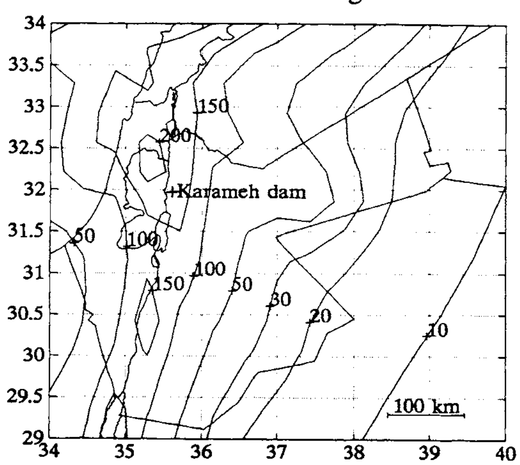



Figure 1: Maximum peak ground acceleration (cm/sec$^2$) with a 50% probability of not being exceeded in 100 years.

The maximum design earthquake (MDE) will produce the maximum level of ground motion according to which the dam should be designed or analyzed. The MDE will be represented by the earthquake that is determined by using the probabilistic procedures with a 50% or higher probability of not being exceeded in a large number of years. During such seismic activity the water-retention capacity of the dam is to be safely maintained. The ICOLD guidelines do not define the "large number"; whereas the United Kingdom (UK) guide, which is largely based on the ICOLD principles, assigns various return periods for the Safety Evaluation Earthquake (SEE) according to dam category or risk class. The SEE is defined as the earthquake which produces the most severe level of ground motion under which the safety of the dam should be insured. The SEE is equivalent to the MDE at the design stage, and depending on dam category, and may be based on a return period. The return period quoted for SEE for Karameh dam (Risk Class II dam) is 3000 years, representing an annual probability of exceedance of 0.00-033. The resulting MDE bedrock acceleration applicable to this exceedance is 0.74g (Figure (2)). In dams design, a value of ⅓ to ⅔ of the maximum peak ground acceleration is used. Therefore, the analysis in this paper was carried out based on a bedrock acceleration of 0.50g through which the water-retention capacity of the dam should be safely maintained.

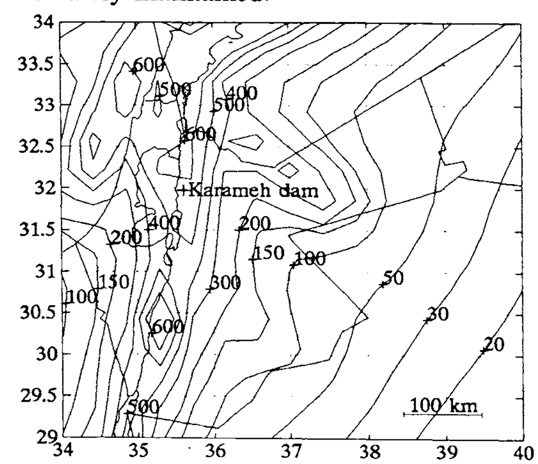
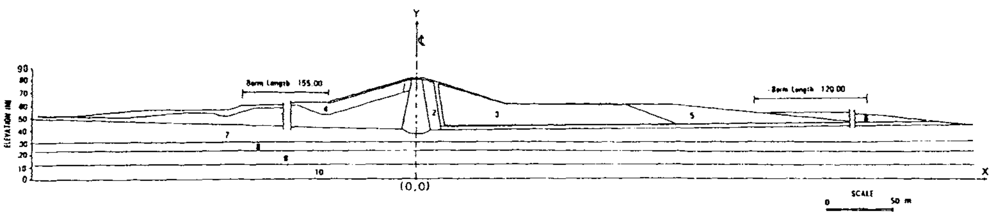



Figure 2: Maximum peak ground acceleration (cm/sec$^2$) with 50% probability of not being exceeded in 3000 years.

## GEOMETRY AND MATERIAL PROPERTIES OF THE DAM

The Karameh dam is zoned earth fill dam with central clayey core. It is presently under construction on a small tributary terminating in the Jordan river near Karameh town. The dam is to be 41.0 m high, 800.0 m wide at base and the storage capacity is 55.0 million cubic meter (MCM). A typical cross section in the middle of the dam, and a borehole log are illustrated in Figure (3). The design material properties for embankment dam and foundation are given in Table (2).



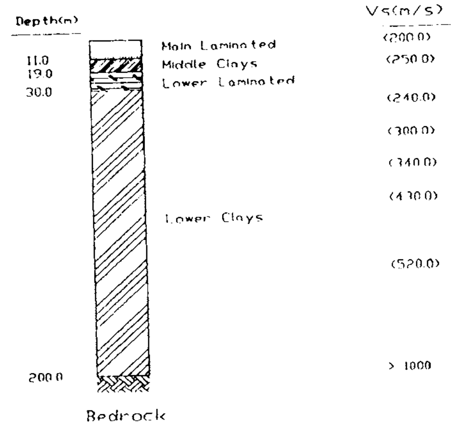




Figure 3: A typical cross section in Karameh Dam, along with a soil column at dam axis.

Table 2: Summary of principal design parameters for Karameh Dam.

| Material | Effective Stress | | Total Stress | | Density | |
|---|---|---|---|---|---|---|
| | c' Kpa | $\phi$' deg. | c' Kpa | $\phi$' deg. | Bulk t/m$^3$ | Saturated t/m$^3$ |
| 1- Core Zone | 5.0 | 29.0 | 20.0 | 0.0 | 1.80 | 1.80 |
| 2- Filter zone | 0.0 | 38.0 | - | - | 2.10 | 2.20 |
| 3-Rolled Fill | 5.0 | 30.0 | 80.0 | 0.0 | 1.82 | 1.91 |
| 4-Shoulder zone | 0.0 | 38.0 | - | - | 2.10 | 2.20 |
| 5-Rolled Fill | 10 | 35.0 | 200.0 | 0.0 | 1.80 | 2.10 |
| 6-Random Fill | - | - | 16.0 | 0.0 | 1.80 | 1.80 |
| 7-Main Laminated | 0.0 | 27.0 | 55.0 | 30.0 | 1.59 | 1.70 |
| 8-Middle Clay | 0.0 | 22.0 | 70.0 | 30.0 | 1.80 | 1.80 |
| 9-Lower Laminated | 0.0 | 22.0 | 100.0 | 65.0 | 1.69 | 1.69 |
| 10-Lower Clay | 0.0 | 25.0 | 175.0 | 130.0 | 1.83 | 1.83 |

## SLOPE STABILITY ANALYSIS

The analysis of the behavior of the embankment under seismic loading was first carried out by using the program REAME (Rational Equilibrium Analysis of Multi-Layered Embankments, Huang 1982) to locate the critical failure circular surface. The method proposed by Sarma (1979) was then used to calculate the critical acceleration factor for the failure surface. Also, the method proposed by Newmark (1965) was used to predict the deformation due to earthquake. Stability analysis has been carried out for the highest central section of the embankment. Five principal design conditions have been considered as shown in Table (3). For each studied case the critical acceleration, the static factor of safety, and the permanent displacement are evaluated.

Table 3: Results of Slope Stability Analysis.

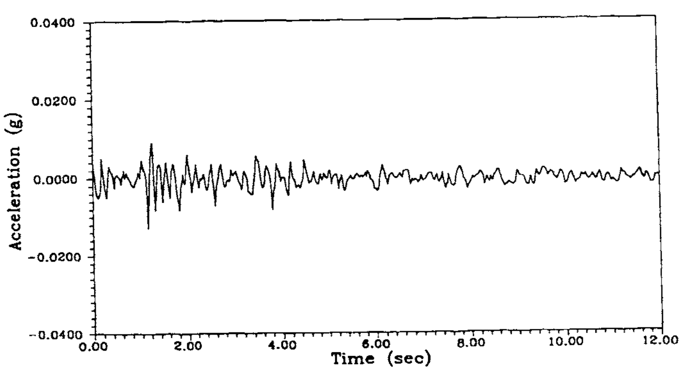
| Case No. | Condition | X - Coord. | Y - Coord. | Radius (m) | Factor of Safety | Critical Acceleration (g) | Displacement (El-Centro) (cm) | Displacement (Dead-Sea) (cm) |
|---|---|---|---|---|---|---|---|---|
| 1 | End of Construction (D.S) | 90.0 | 154.0 | 125.0 | 1.35 | 0.27 | 66.0 | 12.0 |
| 2 | End of Construction (U.S) | -90.0 | 154.0 | 125.0 | 1.68 | 0.14 | 135.0 | 25.0 |
| 3 | Steady Seepage (D.S) | 137.0 | 339.0 | 335.0 | 2.44 | 0.25 | 64.0 | 12.0 |
| 4 | Steady Seepage (U.S) | -50.0 | 119.0 | 55.0 | 3.0 | 0.24 | 46.0 | 9.0 |
| 5 | Rapid Drawdown (U.S) | -130.0 | 234.0 | 210.0 | 2.23 | 0.26 | 53.0 | 10.0 |

## EFFECT OF LOCAL SOIL PROPERTIES ON BEDROCK ACCELERATION

When bedrock is buried beneath an overlying sediments the computed acceleration will not be the same as the acceleration at surface, but will be different in value. The ground acceleration experienced at the top surface of such overlying sediments will depend on the site geology, the properties of the sediments affecting the transfer of ground motion through them and three dimensional influences, particularly the valley geometry.

Several methods for evaluating the effect of local soil conditions on ground response during earthquakes are available at the present time. Most of these methods are based on the assumption that the main responses in a soil deposit are caused by the upward propagation of shear waves from the underlying rock formation. Numerical procedures based on this concept, incorporating non-linear soil behavior, have been shown to give the results in good agreement with field observations in a number of cases. Accordingly, these numerical procedures are used extensively in earthquake engineering for predicting the responses of soil deposits during an earthquake event.

A well proven and widely used 1-D computer code "SHAKE" (Schnabel, et al 1972) was used to numerically model wave propagation through the dam foundation system when subjected to dynamic loading for operating basis earthquake (OBE). The soil deposit is idealized as a series of horizontal layers of infinite extent resting on elastic half space. Each layer is homogeneous and isotropic. The program uses a pseudo non-linear soil model in which the damping increases and shear modulus degrades with the increasing cyclic strain. The acceleration time histories used in the analyses are the 1979 dead sea earthquake (Kfar-Etziyon E-W) ($M_l$=5.1) and the 1940 imperial valley earthquake (El-Centro N00S). The time histories of earthquakes and the response of bedrock acceleration on soil column is shown in the response spectrum represented in Figures (4 and 5). Figure (6) illustrates the spectral amplification ratio (soil/rock) from both the Dead Sea and El-Centro earthquake components.



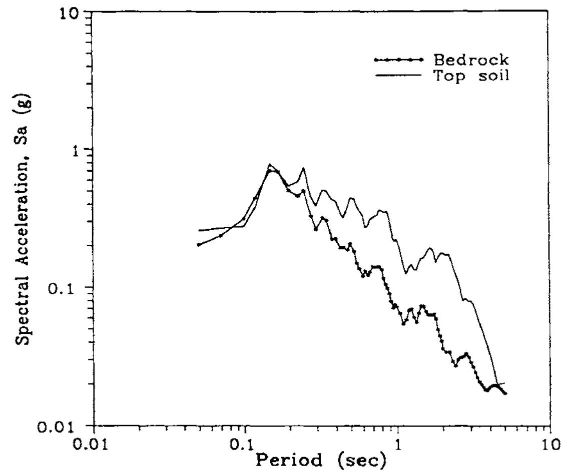




Figure 4: Acceleration time history for 1979 Dead Sea (Kfar-Etzyion) earthquake and response spectra on rock and on top of soil for Karameh dam site. (5% damping)

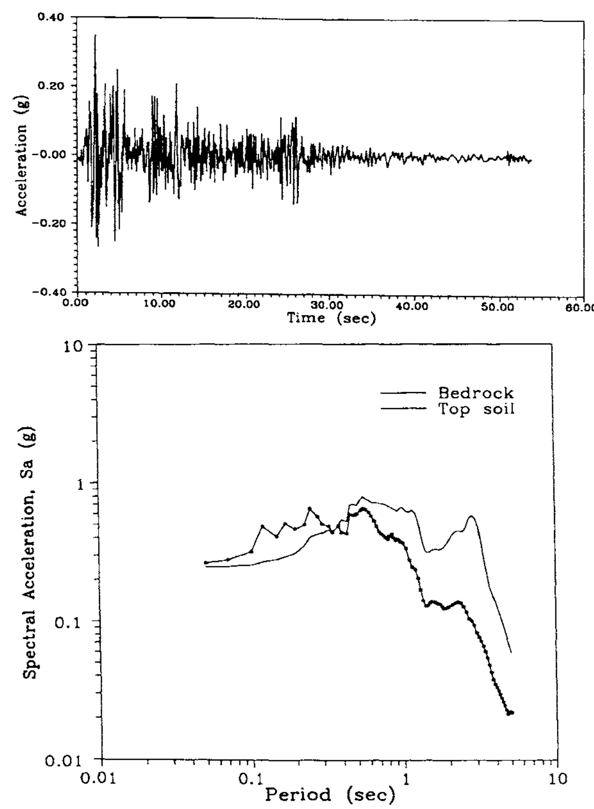



Figure 5: Acceleration time history for El-Centro (N00S) earthquake and response spectra on rock and on top of soil for Karameh dam site. (5% damping)

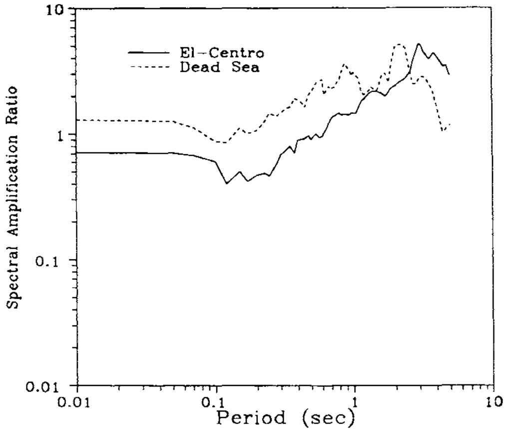



Figure 6: Spectral amplification ratio (soil/rock) under the main dam axis (5% damping).

The effect of change in shear wave velocity of Ghor El-Katar layer (the layer directly beneath the deposit) on the surface acceleration at the base of the dam is evaluated using "SHAKE" program. The results of this analysis are presented in Figure (7). Also, the effect of bedrock depth on the acceleration at the top of soil column is studied using the same model for soil column and the El-Centro (N00S) earthquake component. The results are plotted in Figure (8).

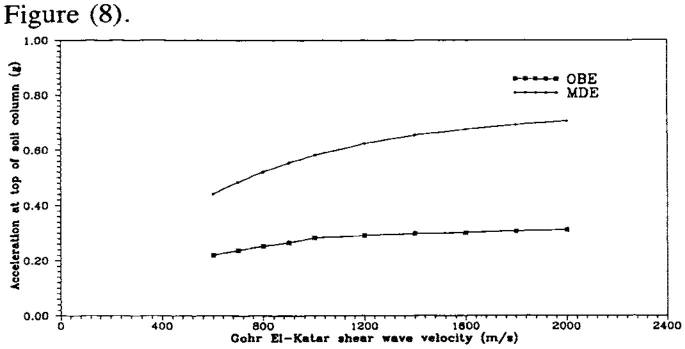



Figure 7: Effect of change in shear wave velocity of the layer beneath soil column on the surface acceleration at the base of the dam, using El-Centro (N00S) component. (OBE), respectively.

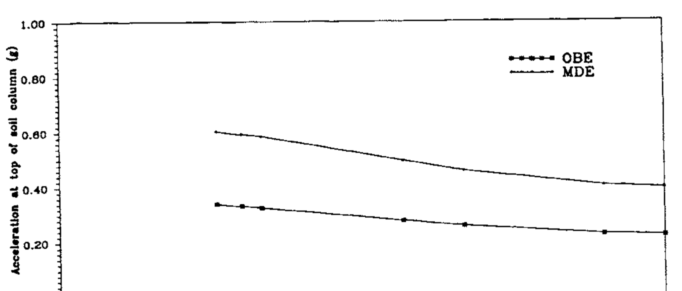



Figure 8: Effect of bedrock depth on surface acceleration of the Karameh dam using El-Centro (N00S) component.

## LIQUEFACTION CONSIDERATIONS AT KARAMEH DAM SITE

Loose, saturated, non-cohesive soils may liquefy when subject to dynamic loading due to an earthquake or other forms of vibratory loading. Liquefaction is related to a rise in the pore water pressure caused by cyclic strain. A threshold strain is needed to produce liquefaction. The particle size distribution for the most easily liquefiable soil is between 0.07 and 0.6 mm. Sands are, therefore, the most susceptible; an increased fines content however, reduces the tendency to liquefaction. The boundary for potentially liquefiable soil is sometimes said to be between 0.02 mm and 2.0 mm. Liquefaction generally not likely to occur in dense sand because of their tendency to dilate, and it is less likely to occur in gravelly soils because of there greater permeability.

The Main Laminated Unit found under the dam site contains some layers of silty, fine sand up to 500 mm thick and, in the Middle Clay Unit, layers of fine to medium sand, also up to 500 mm thick have also been recorded during comprehensive investigations carried out by various geotechnical companies. The sand layers were found to be laterally discontinuous. However, exploratory work in the cliffs west of the dam site has revealed continuous bands of sand approximately 200 mm thick at the top and bottom of Middle Clay Unit. It is, therefore, considered far possible that bands of sand may, in some areas, extend beneath the dam site itself. High pore water pressure generated during seismic events could trigger liquefaction or partial liquefaction of sand layers.

The basic evaluation procedure is based on Seed's (1979) approach which involves a determination of the cyclic shear stresses induced by the earthquake ground motions at different depths in the deposits and conversion of the irregular stress histories to equivalent number of uniform stress cycles.

After several calculations of the possibility of liquefaction in the foundation layers of both the upstream and downstream dam body. It is evident that sand layers in the lower laminated formation are likely to be subjected to triggering. Accordingly, the static stability of the dam must be checked using "undrained" residual shear strength of sand layers after triggering. The dynamic inelastic residual movement due to the remainder of ground shaking after triggering could be upper bound estimate based on Newmark (1965) permanent displacement evaluation method. Using this method, the total spreading due to upstream and downstream slopes could easily exceed 4.4 m. Thus, the crest settlement due to embankment spreading could be a bout of 4.4 m.

## CONCLUSIONS

Based on this study, the following recommendation should be considered:

1) Due to the possible spreading of the dam under earthquake event along with the wave effect and the possible fault movement, the free board should exceed 7.0 m.
2) The foundation of the dam should be improved to avoid any possible liquefaction under the dam site.
3) The performance of the dam during and after the construction should be carefully monitored.

## ACKNOWLEDGEMENTS

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