The 11th Canadian Conference on Earthquake Engineering
Canadian Association for Earthquake Engineering

COMPARISONS OF PSEUDO DYNAMIC TEST RESULTS AND NUMERICAL RESULTS OF AN RCC DAM MODEL

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ABSTRACT: Inspired from the simplified single degree of freedom modeling approach used in the preliminary design of concrete gravity dams, a single degree of freedom pseudo dynamic testing method was devised for the seismic testing of a concrete gravity dam section. The test specimen was a 1/75 scaled section of the 120 m high monolith of the Melen Dam, one of the highest concrete gravity dams to be built in Turkey. First, the single degree of freedom idealization of the dam section was validated in the first stage of the study using numerical simulations including the dam-reservoir interaction. Afterwards, pseudo-dynamic testing was conducted on the specimen using three ground motions corresponding to different hazard levels. Lateral displacement and base shear demands were measured. The crack propagation at the base of the dam was monitored with the measurement of the crack widths and the base sliding displacements. A smeared crack finite element model of the test structure was analyzed for the prescribed ground motions. Load-deformation responses and crack patterns of the test and simulation results were compared. It was found that global response parameters such as displacements and forces were in reasonable agreement whereas accurate estimation of the actual crack lengths was found to be a challenging task.

1. Introduction

Throughout human history, communities have struggled to reach water resources in order to sustain their lives. This effort is presently even more challenging due to the excessive human population and global warming, which necessitates more water storage for both agricultural and energy generation purposes. Definitely, dams are one of the most suitable remedy for this problem. However, the seismic behaviour of dams is very complex due to the interaction of dam body with the flexible foundation rock and compressible impounded water in reservoir, which is proved to be frequency dependent due to the possibility of waves carrying energy away from the dam.

In literature, the first attempts to reveal the dam behaviour under seismic events were to estimate the hydrodynamic forces. These efforts were mainly concentrated on increasing the lateral forces and decreasing the fundamental period of the dam body by simply adding extra masses to the dam structure. Westergaard (1933) was firstly formulated this idea in a complete manner. Kuo (1982) carried Westergaard's idea one-step further by including the formulations for inclined upstream faces. Then, Chopra and his colleagues developed other methods utilizing finite element formulations defined in frequency-domain in order to ease the mathematical complexities based on the frequency-dependent hydrodynamic and foundation effects on the dam structure (Chopra 1970, Dasgupta and Chopra 1979, Fenves and Chopra 1984, Fok et al. 1986, Tan and Chopra 1996 and Chopra and Wang 2008). In other studies, Fenves and Chopra (1985a, 1985b and 1987) proposed simplified methods to simulate the behaviour of dam body under the effect of earthquake excitations based on single-degree-of-freedom response. These studies have been a source of inspiration for the recently published researches (Bouaanani ve Perrault 2010 and Aldemir et al. 2015). In all of these aforementioned researches, the nonlinear behaviour of concrete was neglected for the sake of mathematical simplification. Lately, the safety evaluation of existing dams appeared to be more of a concern in many countries (Gogoi and Maity 2005 and Mills-Bria et al. 2008). On the other hand, the seismic design issue still remains as an important challenge in the countries where new dams are being built such as China and Turkey among others.

Although there are numerous amounts of analytical researches to clarify the seismic behaviour of dams, the experimental studies are very limited in literature due to the difficulties encountered during experimental simulations, i.e. inclusion of hydrodynamic effects, immense geometrical properties, etc. In this regard, engineers usually resort to scaled testing by either changing the density of the dam material (Donlon and Hall 1991) or by adjusting the gravitational acceleration using centrifuges in order to simulate the vertical stresses (Harris et al. 2000) properly according to the similitude laws. Earthquake loading is usually imposed on scaled models using shake tables (Harris et al. 2000 and Uchita et al. 2005). One of the most important disadvantages of the shake table tests is the small duration of the ground motion period due to the (time) scaling of the motion: extremely short testing durations result in significant difficulties in the measurement and observation of the seismic response and the crack propagation within the specimen.

Consequently, in this study, a pseudo dynamic test on a scaled roller compacted concrete (RCC) gravity dam is carried out and the observed crack patterns along with the base shear and crest displacement demands of the dam under three different hazard levels are presented. The selected specimen is 1/75 scaled version of Melen Dam, the highest RCC dam designed in Turkey. Finally, the capability of available finite element models in simulating the seismic response and in forecasting the crack pattern of the selected RCC gravity dam is also investigated.

2. Seismicity and Test Procedure

2.1. Local Seismicity

Melen Dam is located in the first seismic zone, i.e. the most seismically active region defined by Turkish Earthquake Code (TEC 2007). For this purpose, three different design earthquake levels were selected: Operational Based Earthquake (OBE), Maximum Design Earthquake (MDE) and Maximum Characteristic Earthquake (MCE). The site specific design spectrums of these earthquake scenarios are shown in Fig. 1. The site specific spectrums cannot be utilized when performing pseudo-dynamic tests. Thus, spectrum compatible acceleration time histories were also generated (Akkar 2010). These synthetic ground motions along with the site specific spectrums for the most critical governing seismic events are presented in Fig. 1.

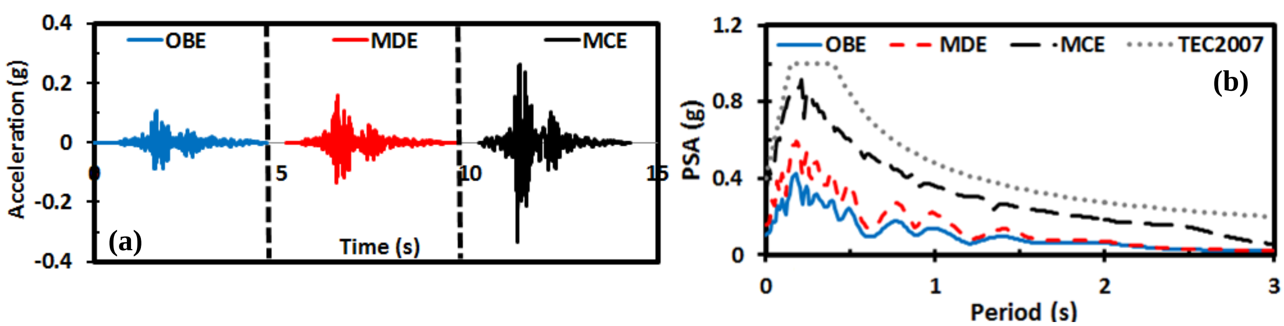


Fig. 1 – Seismicity at the Melen Dam Location : (a) Design Spectrum Compatible Ground Motions and (b) Unscaled Response Spectra for the OBE, MDE and MCE Design Levels

2.2. Summary of Test Procedure

The pseudo-dynamic testing procedures are appropriate for lumped mass systems as they are implemented by utilizing hydraulic jacks at a finite number of joints. Therefore, for distributed mass systems like dam monoliths, the application of the pseudo-dynamic testing procedures requires some simplifications on the test procedure. In literature, Fenves and Chopra (1985) proposed a simplified single degree of freedom (SDOF) approach to estimate the seismic demands on the dam monoliths approximately and, nearly thirty years later, Basili and Nuti (2011) claimed that the single-degree-of-freedom approach was sufficiently accurate in estimating the base shear and overturning moment demands at the dam base. Inspired from these ideas, a pseudo-dynamic testing procedure was proposed in a recent study (Aldemir et al. 2015). For this purpose, the scaled cross-section of a dam is idealized as a distributed stiffness system up to its critical height (h_p) where a concentrated mass (m) is located as shown in Fig. 2. With a proper selection of m and h_p, earthquake induced stress demands at the base of the dam can match those obtained from a rigorous procedure, hence a SDOF idealization is shown to be a viable option for conducting pseudo-dynamic tests of dam monoliths under seismic loading. In this study, the program EAGD, developed by Fenves and Chopra (1984), was utilized to obtain the exact stress and force distributions. In this software, dam-water interaction, wave absorption at the reservoir boundary, water compressibility, and dam-foundation rock interaction are considered by utilizing frequency domain analysis.

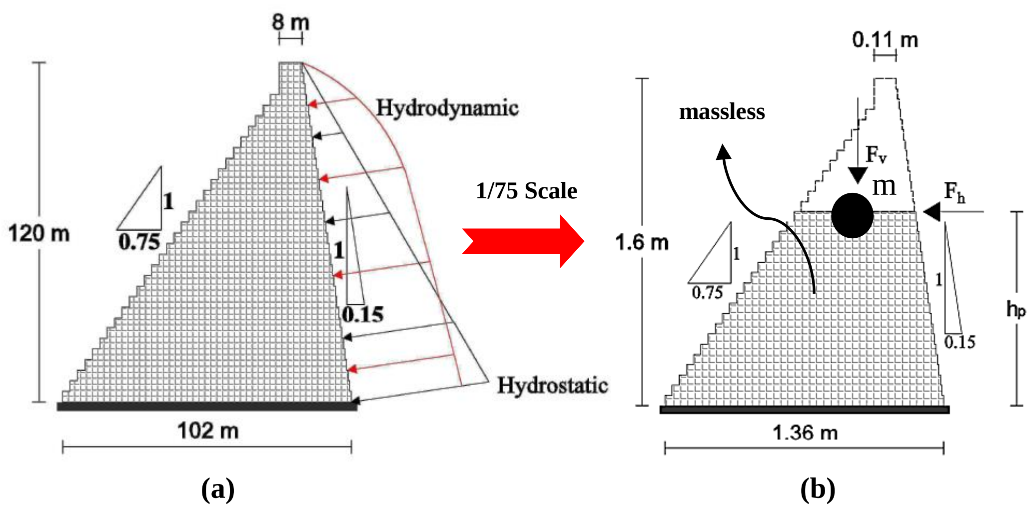


Fig. 2 – Models used in validation of PsD Tests : (a) Model 1 and (b) Model 2

The geometrical properties of the deepest section of scaled and unscaled Melen dam are shown in Fig. 2. As mentioned above, the objective was to obtain analogous stress distributions over the dam base, which was the most vulnerable to cracking portion of gravity dams. At first, the additional physical loads (both vertical and lateral) should be calculated in order to obtain similar stress distributions on scaled dam base. To determine the extra static loads (dead+hydrostatic), a trial-and-error procedure is carried out. In this procedure, the aim is to obtain equal stress distributions over dam base from both scaled (ANSYS) and unscaled (EAGD) models. After this trial-and-error procedure, the closest stress results are obtained with a 400 kN of vertical and 174 kN of lateral loads. It is apparent from Fig. 3.a that the principal stresses over the dam base from both scaled (ANSYS) and unscaled (EAGD) analysis are very similar.

After that, unscaled dam was analyzed using EAGD for the selected ground motions. The resulting overturning moment-base shear response is shown in Fig. 3.b. The slope of the M-V response curve results in h_p=70m, which corresponds h_p=0.95m if a scale factor of 1/75 is applied. This location is roughly the point of application of the resultant hydrostatic force, which enables the use of a single actuator for the prescription of the static and dynamic loads. Scaled dam model was then analyzed with the same ground motions for different values of concentrated mass (m) in order to determine the optimal equivalent mass to use in the analyses. The m values that minimize the difference between the demand parameters obtained from scaled and unscaled models were identified as m_1=37.5 ton, m_2=40.0 ton, m_3=55.0 ton for the OBE, MDE and MCE ground motions, respectively. The comparisons of the base shear, moment and stress demands obtained from the dynamic analyses of scaled and unscaled models for OBE and MCE motions are presented in Fig. 4 and 5. Scaled model estimates the time history of the stress with a reasonable accuracy (less than 20% error for the vertical stresses). More explanation on the test procedure can be found in Aldemir et al. (2015).

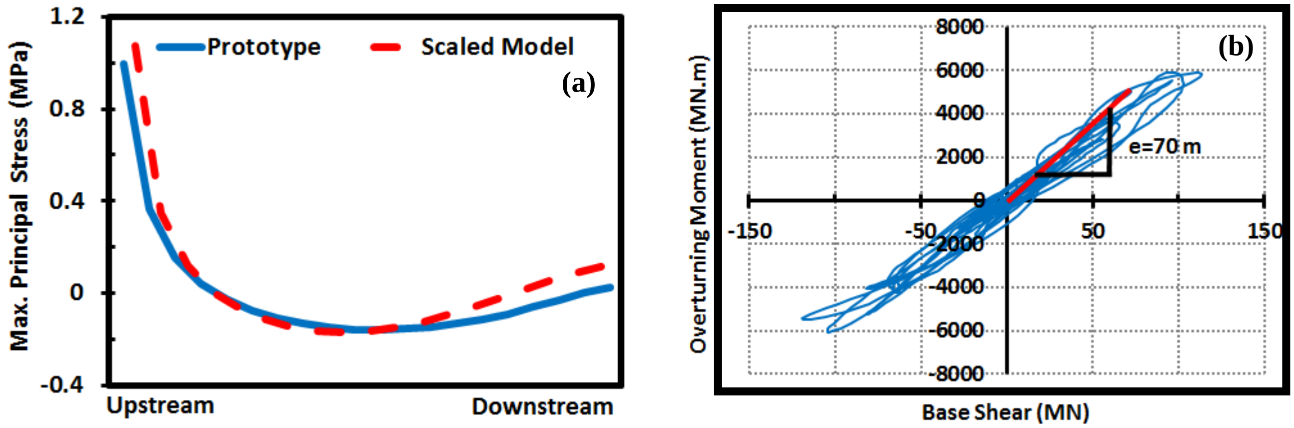


Fig. 3 – ANSYS and EAGD comparisons : (a) Maximum principal stresses from statical loading effects and (b) Determination of effective height

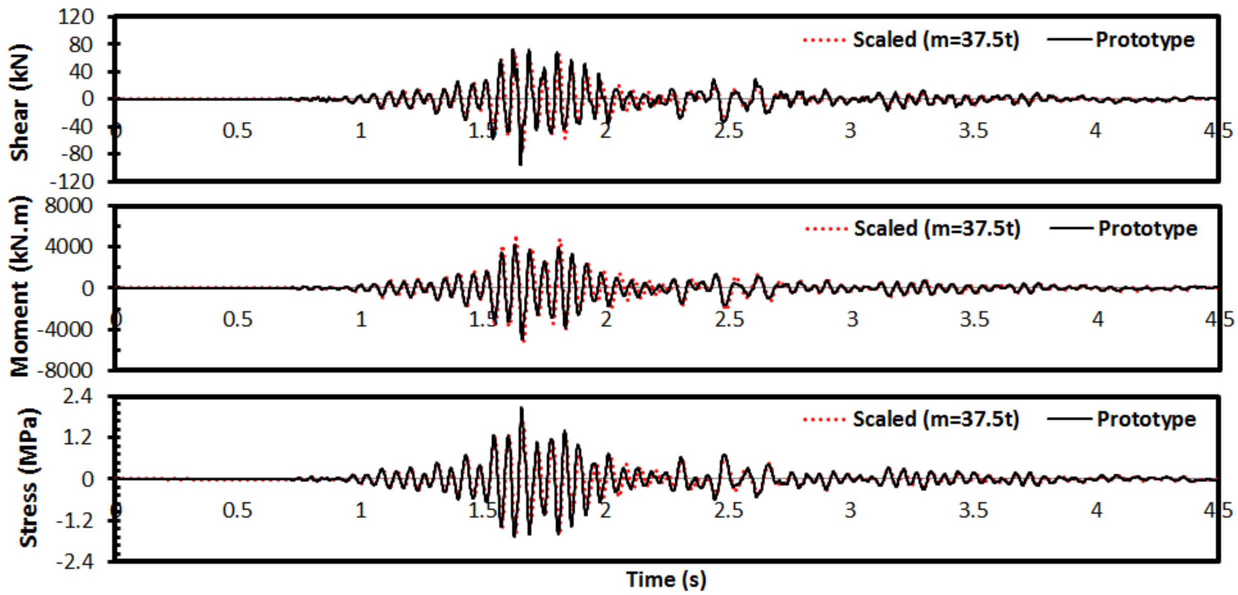


Fig. 4 – Comparison of the Analysis Results for the OBE ground motion: (a) Base Shear, (b) Overturning Moment and (c) Vertical Stress

3. Experimental Results

After the curing of concrete, the formwork was removed and the instrumentation was installed on the test specimen (Fig. 6). Linear variable differential transducers (LVDTs) were used to measure the relative displacement of the top of test specimen with respect to its base. A high precision displacement transducer (accuracy of ±10 μm) was used to provide the displacement feedback of the specimen at the top while executing the pseudo dynamic test. In addition, the displacement at the base of the specimen was measured with respect to the foundation connected to the strong floor. Detailed information on the instrumentation could be found in Aldemir et al. (2015).

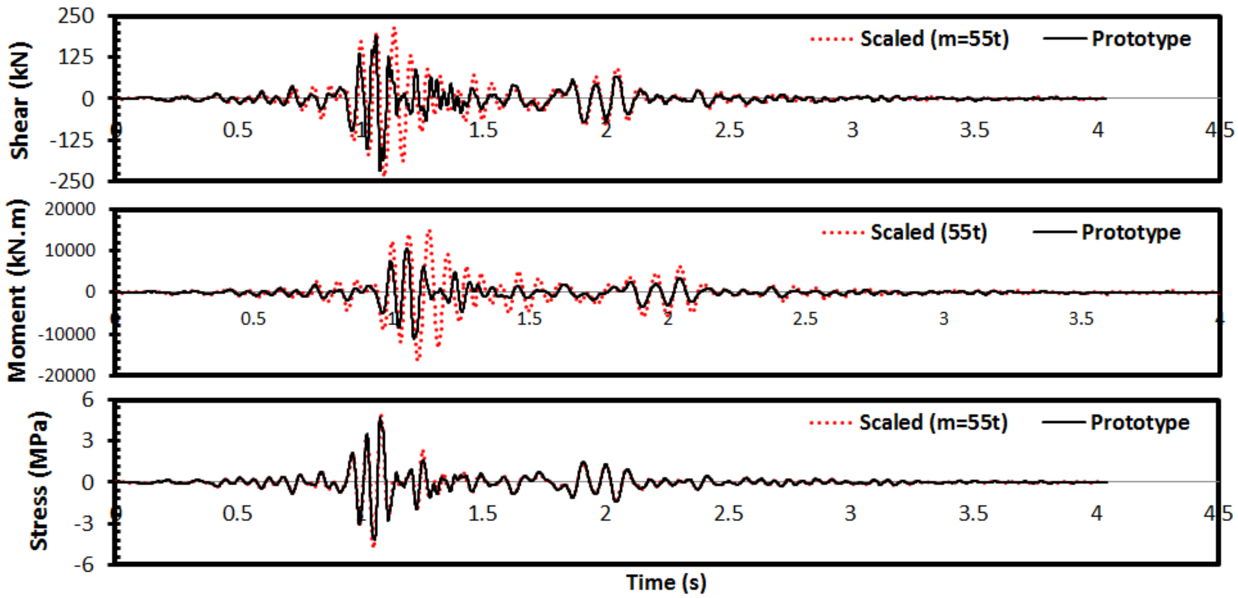


Fig. 5 – Comparison of the Analysis Results for the MCE ground motion: (a) Base Shear, (b) Overturning Moment and (c) Vertical Stress

The mechanical properties of the roller compacted concrete was also determined at the test day. The compressive strength, the split tensile strength and modulus of elasticity were detected to be 23.10 MPa, 2.92 MPa and 21,305 MPa, respectively.

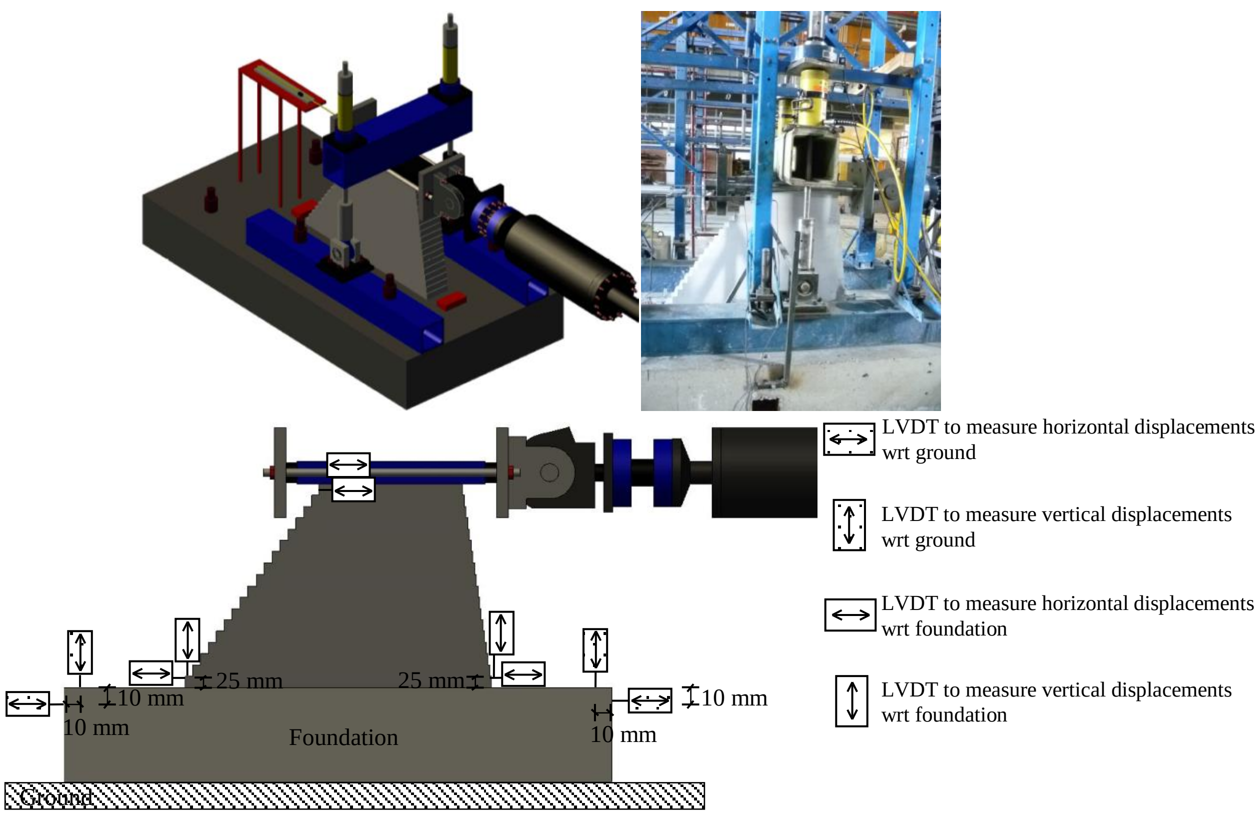


Fig. 6 – Test Setup and instrumentation : (a) LVDT's at upstream dam base, (b) LVDT's at downstream dam base, (c) High precision displacement transducer and (d) LVDT's at the dam tip

The experiment started with the application of a vertical load of 400 kN via tie rods and pistons. After that, the lateral load of 174 kN, simulating the hydrostatic forces, were applied to the specimen via pseudo-dynamic piston and, at this point, the load on the pseudo-dynamic piston and the control displacement (Heidenhain) were zeroed before implementing dynamic loads (One more load cell and LVDT continued measuring the load and tip displacement from the initial position via another data acquisition system). The same specimen was exposed to three different hazard levels: OBE, MDE and MCE, respectively. At each hazard level, the displacement, force demands and damages were observed (Fig. 7). The base shear-tip displacement curves for each hazard level was obtained. However, as these capacity curves are presented while comparing the analytical results with the experimental findings in the next part of this study, they are not given in this part due to the length limitations.

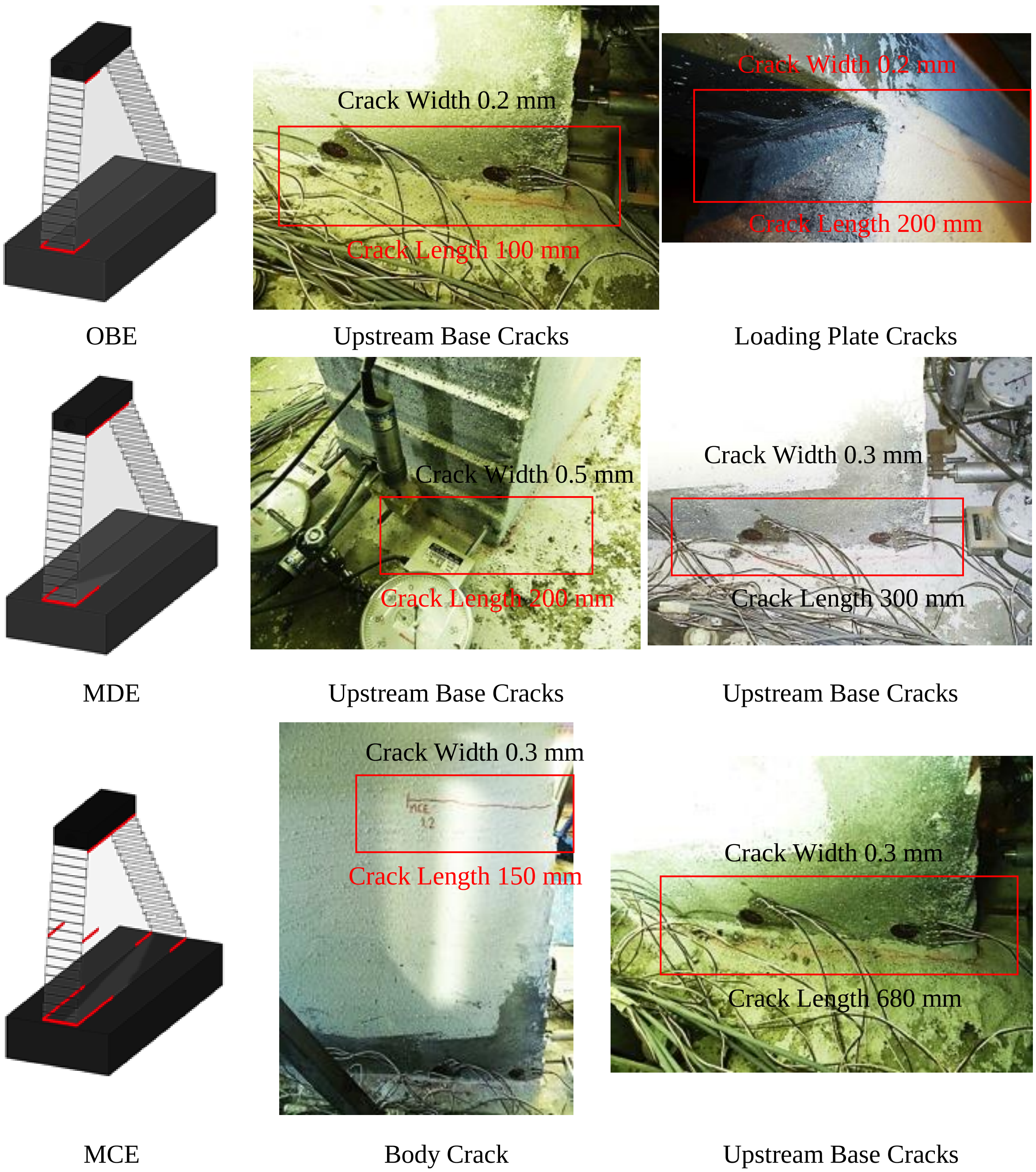


Fig. 7 – Observed Cracks during OBE, MDE and MCE Experiments

4. Analytical Results

3D model of the specimen was formed in the commercially available software ANSYS (2007). The failure criterion Willam – Warnke (1980) was utilized to simulate the cracking and crushing of concrete (Fig. 8.a). To be compatible with the Willam – Warnke plasticity model, 8-node solid elements (Solid 65) were used. The detailed information on the utilized plasticity model and the solid element formulations could be found from the theoretical manual of ANSYS. The base nodes of the dam specimen were assumed to be fixed for any translational effects (Fig. 8.b). Although a thick concrete foundation was utilized to simulate a fixed base for the specimen, during experiments, some vertical motion (in the order of 1mm) was detected. Therefore, the effect of base rotation on the fundamental vibration period of the test specimen was reflected to the analytical model by reducing the modulus of elasticity by approximately 50% of the measured one. The amount of reduction was determined by matching the identified fundamental period at the beginning of the first test (OBE) with the one obtained from the analytical model. In addition, the identified damping ratios were utilized in the time history analysis for each hazard level.

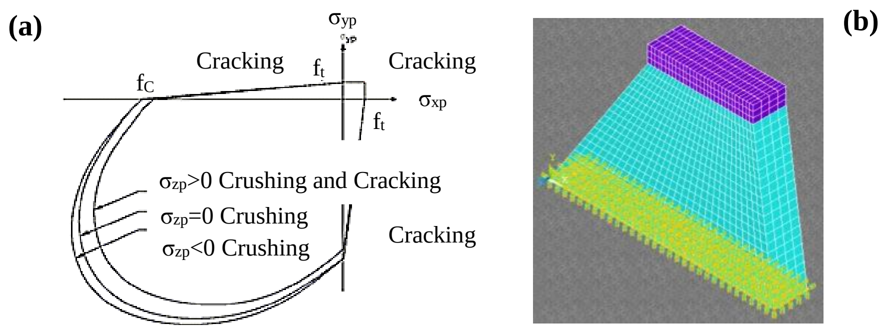


Fig. 8 – ANSYS Models : (a) Willam – Warnke Model and (b) Finite Element Mesh

The same loading protocol was utilized during the analytical model as the experimental one. In other words, firstly, the specimen was loaded vertically and then it was pushed by a lateral load of 174 kN to simulate the hydrostatic effects. Finally, the earthquake ground motions for each hazard level were applied to the specimen consecutively. The base shear and tip displacement curves from analytical model were compared with the experimental ones (Fig. 9). Moreover, the ability of the numerical model to estimate the crack patterns were also investigated (Fig. 10). Finally, the full capacity curve of the dam specimen was determined by conducting a pushover analysis (Fig. 11) to the damaged specimen (after MCE scenario). This procedure was the same as the one implemented during the experimental investigation of the specimen. The percentage errors in the base shear and tip displacement estimations from analytical model are summarized in Tables 1 and 2. From these tables, it is clear that the mazimum base shear could be predicted with an error of less than 30% whereas the error in the maximum tip displacement estimations reaches to 80%. However, the capacity curve of the specimen is predicted very well, implying that the hysteretic behaviour model has some difficulties to simulate the real behaviour. The errors in base shear and tip displacement are less than 10%.

Table 1 – Percentage Errors in Base Shears (Maximum values from experiment are shown in red italics.)

Experiment			ANSYS			Error (%)		
OBE	MDE	MCE	OBE	MDE	MCE	OBE	MDE	MCE
146.258	*176.514*	*307.422*	176.072	213.978	150.167	*-20.385*	*27.481*	*12.739*
-101.10	-103.54	-219.02	-109.93	-128.01	-268.26	-7.778	-106.66	31.436

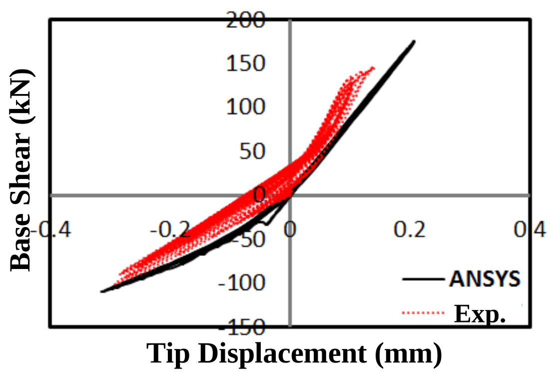
5. Conclusions

In this study, the ability of analytical methods to simulate the experimental behaviour of a roller compacted concrete dam is investigated. The following conclusions can be drawn on the basis of findings of this study.

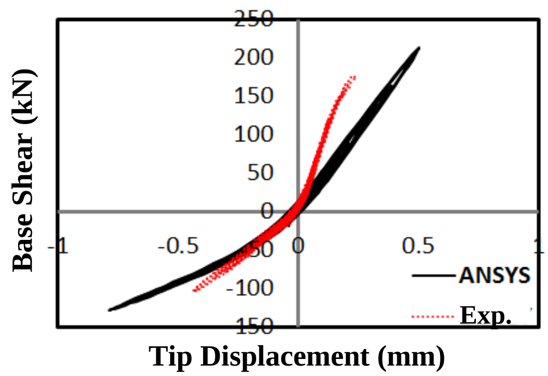
- The finite element model estimated the first crack as the base crack at the upstream face, which was verified by the experimental observation. In addition, both experiment and analytical models showed that the stability of dam specimen was not disturbed under any of the applied earthquake excitations. However, the body crack could not be predicted by the analytical model.
- Commercially available finite element softwares can predict the base shear and tip displacement demands with errors of less than 20% and 35%, respectively. These error ratios are detected to be even lower (less than 10%) in predicting the capacity curves under pushover loading.
- However, the success of the finite element simulations in simulating the crack patterns is questionable as the analytical model estimates more damage than the observed one due to the stress concentrations at the possible crack locations. Therefore, the failure criteria for concrete and stress redistribution at the crack locations should be investigated in detail.
- According to the analysis results, the global parameters like base shear, tip displacements, etc. could be estimated better than the stresses, crack widths, etc. Therefore, at the design stage, it is wise to use global demand parameters rather than stress and crack checks.

Table 2 – Percentage Errors in Tip Dispalcements (Maximum values from experiment are shown in red italics.)

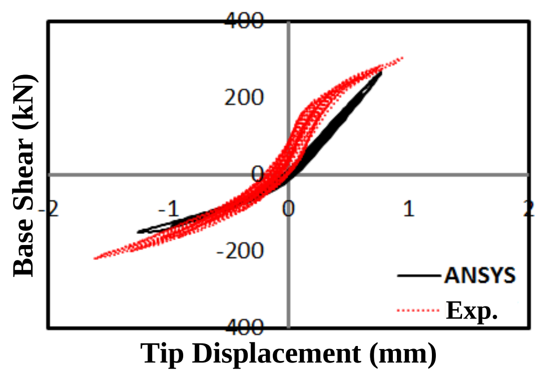
Experiment			ANSYS			Error (%)		
OBE	MDE	MCE	OBE	MDE	MCE	OBE	MDE	MCE
0.140	0.238	0.958	0.208	0.504	0.772	-48.078	-111.86	19.400
-0.294	*-0.435*	*-1.625*	-0.315	-0.783	-1.256	*-6.888*	*-80.041*	*22.662*



(a)



(b)



(c)

Fig. 9 – Comparison of Analytically Estimated Base Shear – Tip Displacement Curves with the Experimental Curves : (a) OBE, (b) MDE and (c) MCE

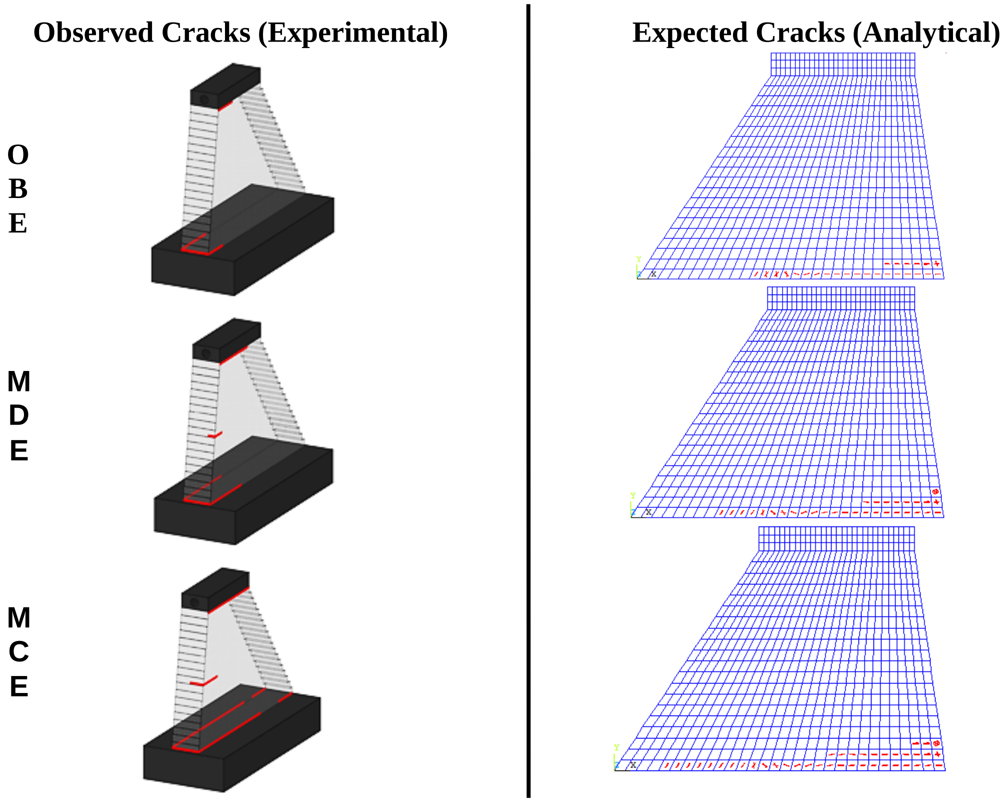


Fig. 10 – Comparison of Expected and Observed Crack Patterns for OBE, MDE and MCE Scenarios

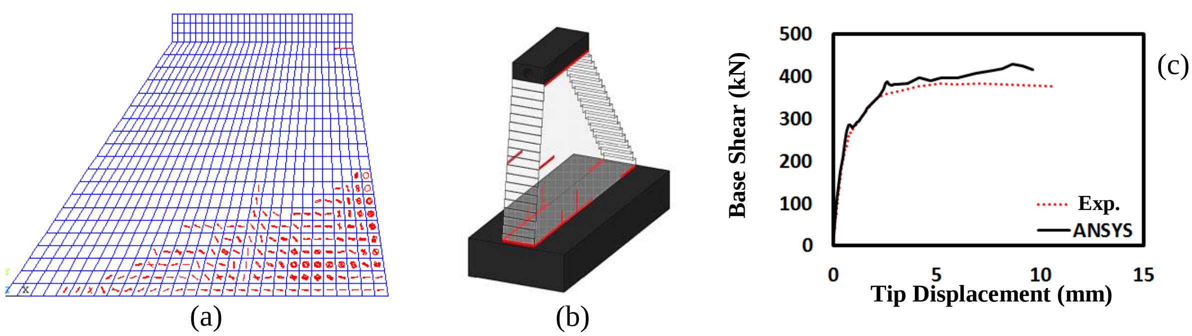


Fig. 11 – Comparison of Results for Pushover Experiment: (a) Expected Crack Pattern; (b) Observed Crack Pattern and (c) Capacity Curve

6. Acknowledgements

This study was supported by the Turkish National Sciences Foundation TUBITAK under the grant number 111M712. The second author acknowledges the support from TUBA-GEBIP for continuing research endeavors. The contributions of Salim Azak and Hasan Metin during the laboratory work are also gratefully acknowledged.

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