Seismic performance evaluation of a concrete gravity dam by using pseudo dynamic testing

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ABSTRACT

There are about 507 dams among 450 of which are embankment dams built in Turkey and Turkey is located in a seismically active region. Therefore, it is well known that the building stock is under great risk. The idea of having a similar risk for the hydraulic facilities is extremely frightening and may endanger the health and wealth of the society. More importantly, the seismic behaviour of concrete gravity dams has not been investigated experimentally in detail other than a few shaking table test programs. In order to develop performance based seismic design rules for concrete dams, the relationship between damage and performance acceptance criteria should be determined. Therefore, the objective of this paper is to summarize the experimental results from a pseudo dynamic test of a scaled concrete gravity dam. The emphasize will be on base cracking, sliding, base shear capacity and crest displacements of the dam in concern under three different hazard levels.

1. INTRODUCTION

The energy sector in Turkey has recently been privatized leading to a large scale investment in hydroelectric power (Dursun and Gokcol 2011). Yet, the building stock in Turkey is in a great risk since nearly all parts of Turkey lies on a seismically very active zone. Therefore, the design of any type of buildings requires the seismic effects to be taken into account. In this context, the seismic safety of large dams is becoming an important issue as the majority of the dam stock resides in seismically active areas.

After the pioneering work of Westergaard (1933) on calculating the hydrodynamic force during the earthquake excitation, seismic analyses and design of concrete gravity dams were studied in detail by Chopra and his colleagues. 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

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remains as an important challenge in the countries where new dams are being built such as China and Turkey among others.

In literature, there are numerous analytical studies on different modeling techniques of dam structures. In some studies, the seismic analysis of dam-reservoir-foundation interaction is generally treated as a two dimensional problem for concrete gravity dams, owing to the existence of transverse joints separating the structure into monoliths (Fenves and Chopra 1984.a and 1984.b). As far as three dimensional analysis methods are concerned, it can be claimed that most of them are developed in reference to arch dams. In other analytical studies, rigorous linear elastic formulations incorporating the frequency dependent behaviour of reservoir and foundation are proposed to investigate the behaviour of arch dams by using three dimensional finite and boundary element methods (Wang and Chopra 2008). In the framework of nonlinear finite element analysis in some studies, the effect of the contraction joint behavior and the cracking of concrete are also treated by using special joint elements and smeared crack models, respectively. Despite the early evaluations of Rashed and Iwan (1984), who pointed out the need of three dimensional analyses for the seismic design of dams in narrow canyons, three dimensional nonlinear finite element analyses are rarely employed in the design of concrete gravity dams. Unfortunately, performance-based earthquake engineering design principles are not included in these studies.

However, the recent innovations in earthquake engineering necessitates the determination of structural performance under seismic effects, expected damage, repair cost estimations after earthquakes, etc. at the design stage. And, it is apparent that all of the above goals require detailed performance prediction methods including good damage-performance relations, correct seismicity predictions, realistic analytical models and the verification of the analytical models.

The experimental studies on examining the performance of concrete gravity dams are very limited. This is because; it is very difficult to test these huge structures in laboratory environment as they require very large scaling, resulting in tremendous amount of external axial load application to simulate the static stress distributions of unscaled dams. At this point, it should be noted that the density of material or the gravitational acceleration should be increased by scale in order to simulate the vertical effects in a realistic way (the same stress distribution at the base of the scaled dam). The first remedy could change the cracking pattern, so the best way is to use centrifuge to increase the gravitational acceleration. This method is also utilized in the experimental study carried out by Uchita *et al.* (2005). In this context, the most important studies are the shaking table tests as they offer the most realistic simulation of earthquakes. In literature, there exist two small scaled Koyna Dam experiments by Donlon and Hall (1991) and Harris *et al.* (2000). In these experiments, different materials were used to obey the rules of similitude but the mechanical properties of those materials were different than concrete. In addition, there were other experiments at Bristol University (1995), at University of Colorado in association with Tokyo Electric Power Service Company (2005) and at Canada (2000, 2002 and 2009). Also, Ghobarah and Ghaemian (1998) were tested a 2D dam section under the effect of cyclic displacement demands and observed the crack propagations. However, one of the most important disadvantage of the shaking table tests is the small duration of earthquake due to the rule of similitude (duration should be decreased by the square

root of the scale). Therefore, it is very difficult to observe the experimental behaviour and to detect the crack propagation of the specimen.

Fenves and Chopra (1984.a and 1985) concluded that 2D dam sections behave first mode dominant under the effect of earthquake. This hypothesis gives birth to the idea that the pseudo-dynamic testing could also be applied to distributed-mass systems like dams, masonry, etc. To accomplish this, the inertial forces and the hydrodynamic effects would be acted on the dam specimen at one point such that the stress distributions at the base of the scaled and unscaled dams are similar during both static and dynamic loading conditions. The objective of the present work is to investigate the behaviour of a concrete gravity dam under the effect of three different ground motions.
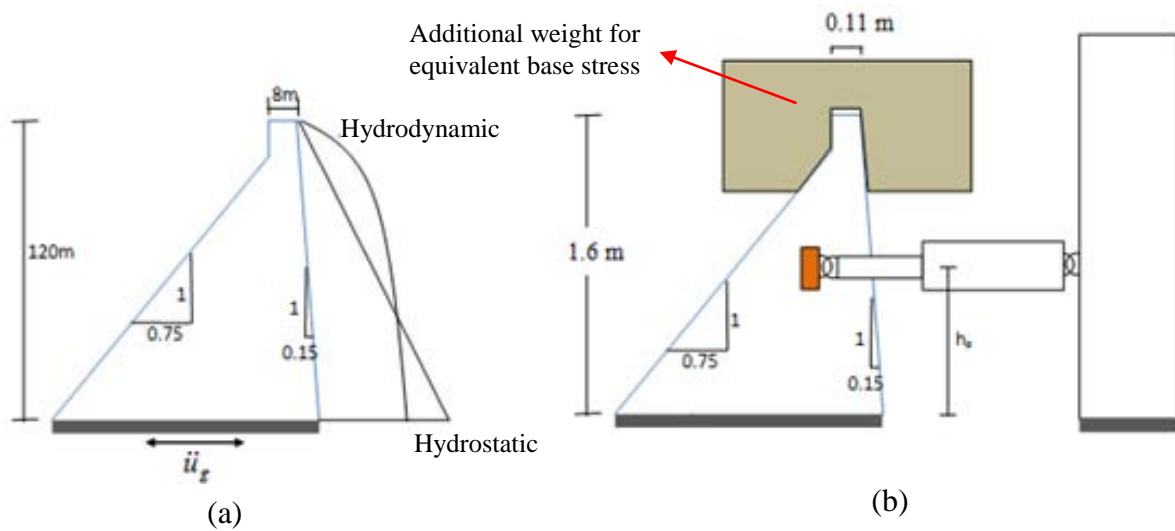

Fig. 1 Geometrical Properties : (a) Deepest Section of Melen Dam and (b) 1/75 Scaled Test Specimen

2. TEST SETUP

Throughout this study, 1/75 scaled version of Melen Dam, designed to supply drinking water and to generate electricity to Istanbul, will be utilized. The geometrical properties of the deepest section of scaled and unscaled dam are summarized in Fig. 1. The main aim is to obtain similar stress distributions over the dam base, which is the most vulnerable to cracking portion of a gravity dam. To accomplish this, the same base shear and the overturning moment should be created via the pseudo-dynamic piston as the distributed mass system solution. The assumptions made for the preparation of test setup are as follows:

- Inertial forces will act at a single point. This point satisfies the equivalent base shear and overturning moments for elastic case.
- All of the tests would reflect the full reservoir conditions.
- The average compressive strength of concrete is 25 MPa at 28th day.

- The specimen shows single mode dominant behaviour. This assumption is in acceptable limits because the expected damage will occur as base cracking and sliding.
- Material behaviour does not change with the loading rate, so pseudo-dynamic testing could be performed at a slow loading rate.
- As the purpose of the experiment is to simulate realistic earthquake demands (base shear and overturning moment) at the dam base, the accuracy of stress distribution within the dam body shall be out of interest. This is because; the observed damage is often base cracking and sliding in dam-type structures (Chopra and Zhang 1991).

3. ANALYTICAL STUDIES

In this part, the analytical studies performed to form the test specimen are described. As the main assumption of the test is single mode dominant behaviour, there will be only one pseudo-dynamic piston. The location of the piston and the additional mass amount are obtained such that the single-degree-of-freedom (SDOF) and the multi-degree-of-freedom (MDOF) systems show similar base demands for static and dynamic loadings. This is accomplished by selecting different pairs of height (h_p) and additional mass (m) and comparing MDOF demands from EAGD and SDOF demands from ANSYS. When the results (stresses and crest displacements) show similarity, then the trial and error procedure could be stopped. At this point, it should be stated that the results from EAGD (Fenves and Chopra 1984.b) are taken as the true solution as this program is capable of taking dam-foundation-reservoir interaction into account. In all of the analyses, 5% damping ratio is utilized. Before giving details of above-mentioned analyses, the local seismic risk results will be explained.
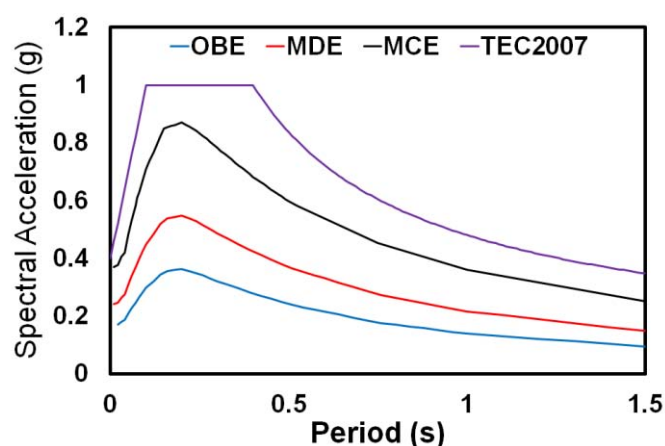

Fig. 2 Site Specific Design Spectrums for OBE, MDE and MCE

3.1. Local Seismic Risk Results

Melen Dam is located in a seismically very active region (the first seismic zone defined by Turkish Earthquake Code 2007 (TEC2007)). The spectrum compatible

ground motions were developed after determining the site specific design response spectrum (Akkar 2010). 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. 2. 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 for the most critical governing seismic events are presented in Fig. 3.

(a)   (b)   (c)

Fig. 3. Site Specific Design Spectrum compatible Synthetic Ground Motions :
(a) OBE, (b) MDE and (c) MCE

3.2. Determination of Additional Loads and Masses

In this part, the additional mass and piston height were determined. To accomplish this, EAGD, developed by Fenves and Chopra (1984.b) was utilized. This program implements frequency domain analysis technique and considers the following crucial factors in the analyses: dam-water interaction, wave absorption at the reservoir boundary, water compressibility, and dam-foundation rock interaction. Therefore, the exact stress, base shear and overturning moment demands could be obtained by EAGD (of course, in linear range). At first, the additional physical loads (both vertical and lateral) should be calculated in order to obtain similar stress demands 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 175 kN of lateral loads. It is apparent from Fig. 4 that the principal stresses over the dam base from both scaled (ANSYS) and unscaled (EAGD) analysis are very similar.
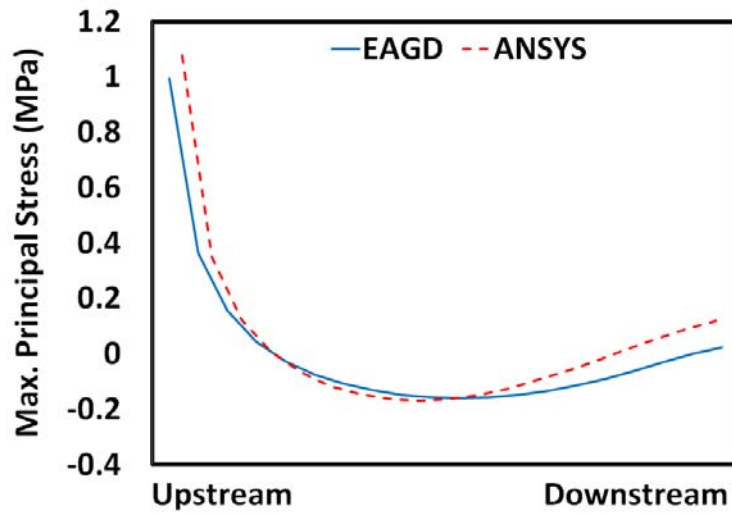
Fig. 4 Comparison of Maximum Principal Stresses from Statical Loading Effects

After that, the exact base shear - overturning moment history of dam specimen are found (by utilizing EAGD). Then, the effective height (piston location) could be determined from the slope of this graph (Fig. 5).
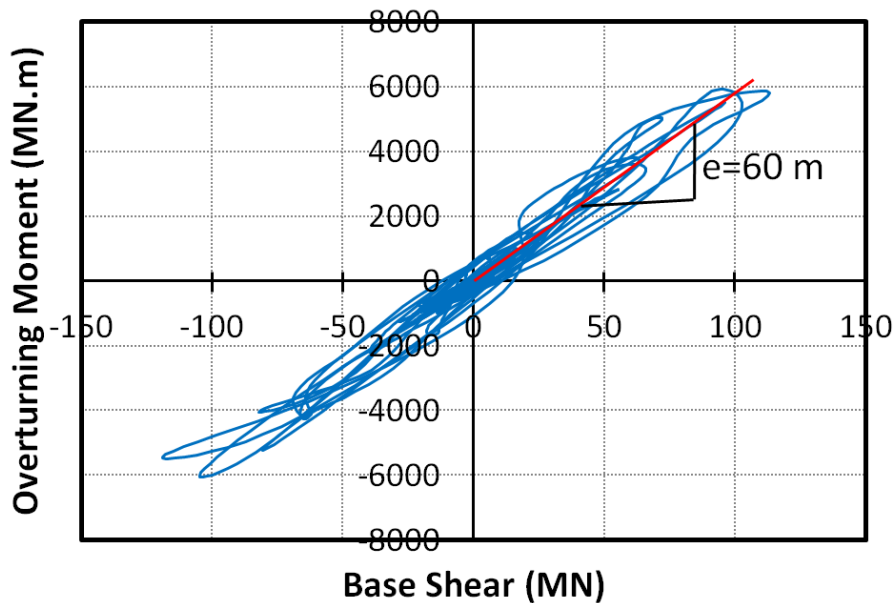

Fig. 5 Determination of Effective Height

After determining the effective height, the additional masses should be found out. This time, the additional masses (for dam body inertia+hydrodynamic effects) are so adjusted that the base shear and the overturning moment responses of both scaled and unscaled dams are close to each other. The calculated additional masses for three different hazard levels are summarized in Table 1. These additional masses are supplied

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to the algorithm of pseudo-dynamic testing system. In , the comparison of base demands (EAGD and SDOF) are summarized. In addition, the comparison of stresses at the upstream toe is shown in 7.

Table 1 Additional Masses

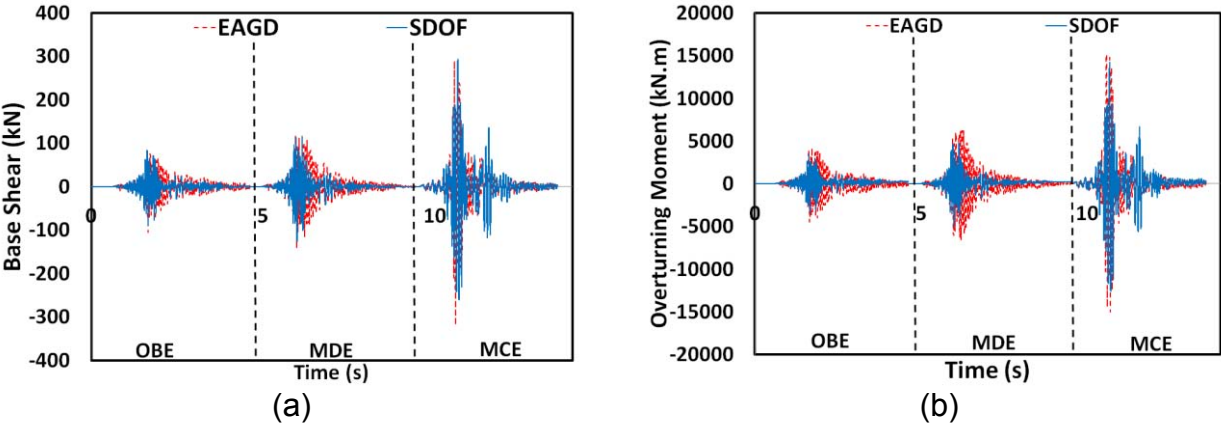| Hazard Level	Additional Mass
OBE	37.5 ton
MDE	40 ton
MCE	55 ton

(a)

(b)

Fig. 6 Comparison of Analysis Results : (a) Base Shear and (b) Overturning Moment

As it can easily be inferred from Fig. 7, SDOF system could estimate the stresses within an error less than 30%. Moreover, this error is below 20% at the maximum hazard level (MCE), which is more important as far as the damage and stability investigations are concerned. More importantly, the principal stresses, which are responsible for the cracking and crushing, are estimated with an error of nearly 10%. This much error is beyond the expectations for stresses, which are known as having one of the lowest accuracy in finite element analyses. In conclusion, SDOF system response is within the acceptable limits in comparison with MDOF ones.

Finally, it should be stated that in the entire above analyses plane stress elements with linear displacement shape functions (4 nodes) are utilized in both ANSYS and EAGD analyses in order to eliminate the effect of formulation on the results.
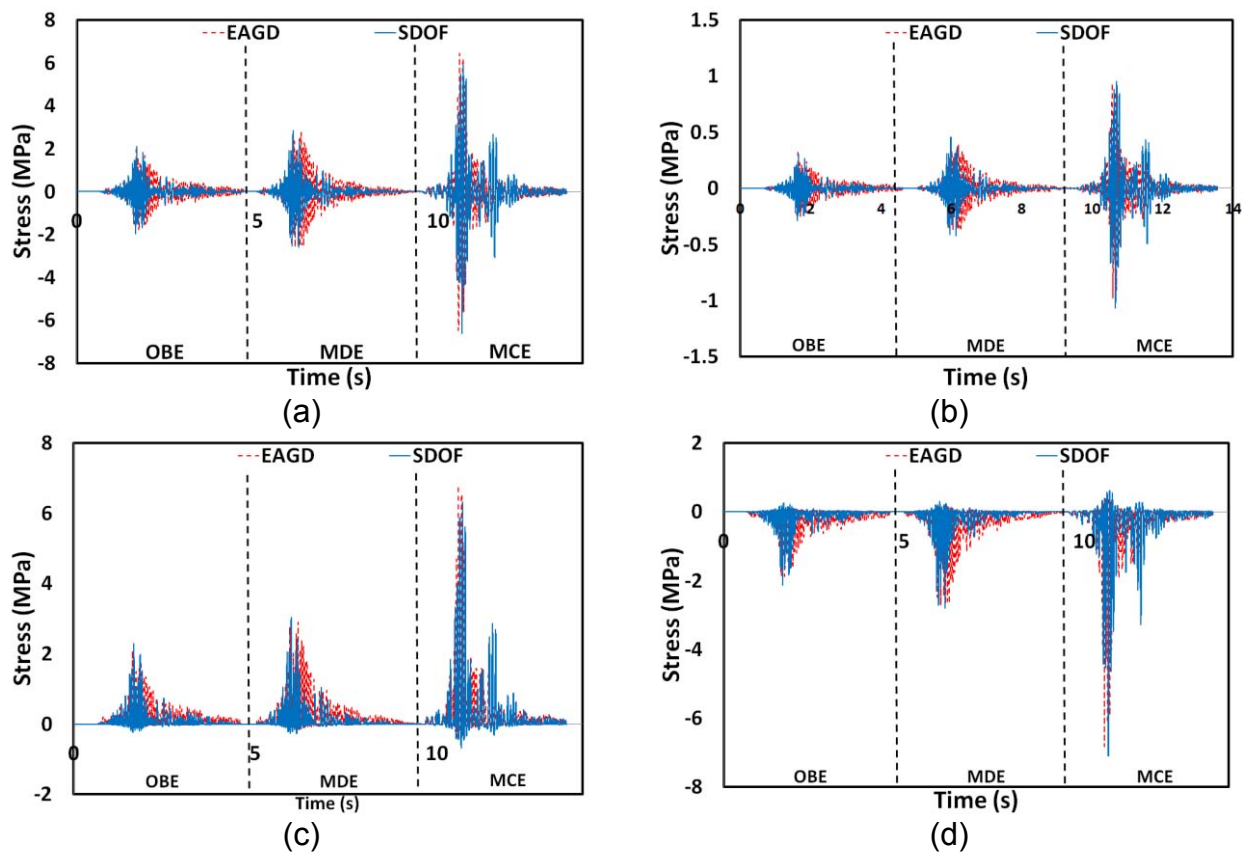
Fig. 7 Comparison of Stresses: (a) Axial Stress, (b) Shear Stress, (c) Maximum Principal Stress (Tension) and (d) Minimum Principal Stress (Compression)

4. EXPERIMENT

4.1. Instrumentation

Concrete curing was continued for seven days. After the curing process of the concrete, the preparation for placement of measurement devices was started. As the displacement feedback (Heidenhain) was placed on the foundation, the slips occurring at the foundation level did not affect the pseudo-dynamic algorithm, i.e. foundation deformations were separated from the equation of motion. Two high capacity LVDT's (50 mm and 100 mm) were placed at the bottom of upstream and downstream faces to measure the probable base slip. However, the accuracy of these LVDT's was maybe not enough in case there was no slip. So, a precaution was taken by placing one more LVDT (10 mm capacity) at the bottom of upstream and downstream faces (Fig. 8). Furthermore, the base rotation was also measured by placing two vertical LVDT's (20 mm capacity) at the base (Fig. 8).
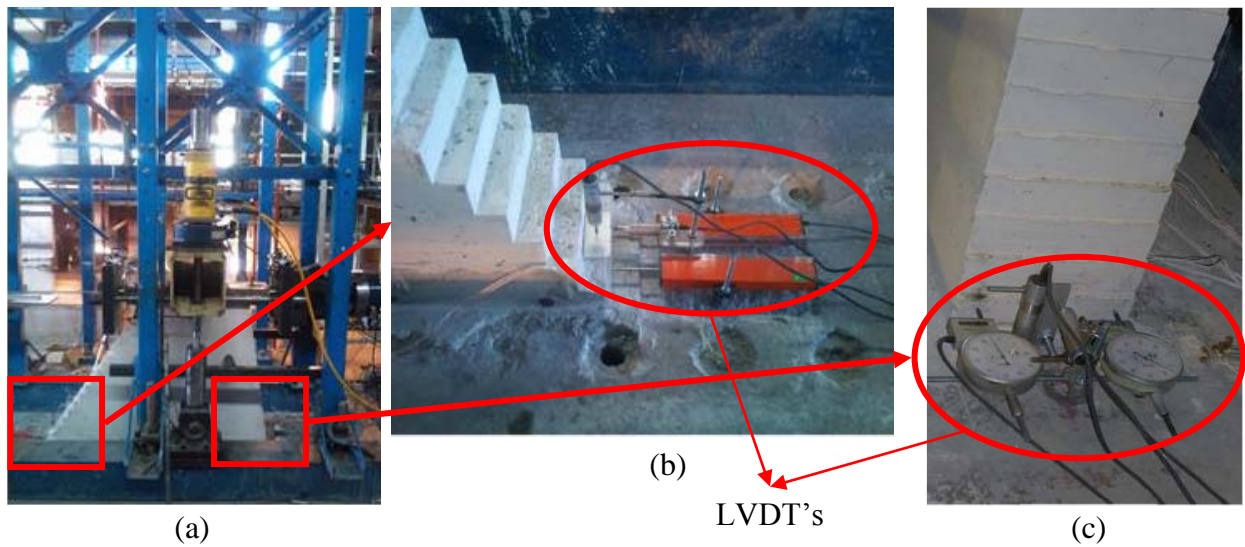
Fig. 8 Dam Base Measurements : (a) Side View, (b) Downstream Face and (c) Upstream Face

Four measurement devices were placed to record the horizontal tip deflection of the dam specimen. The first one was placed on the load transfer plate (Fig. 9). The other LVDT was mounted just under the first one but on the concrete dam body. Therefore, the slip between the transfer plate and dam body could be detected (Fig. 9). Also, the control displacement was supplied by a Heidenhain at the tip of the specimen (Fig. 9). One more LVDT was set just next to the Heidenhain so as to inspect the control displacement accuracy.

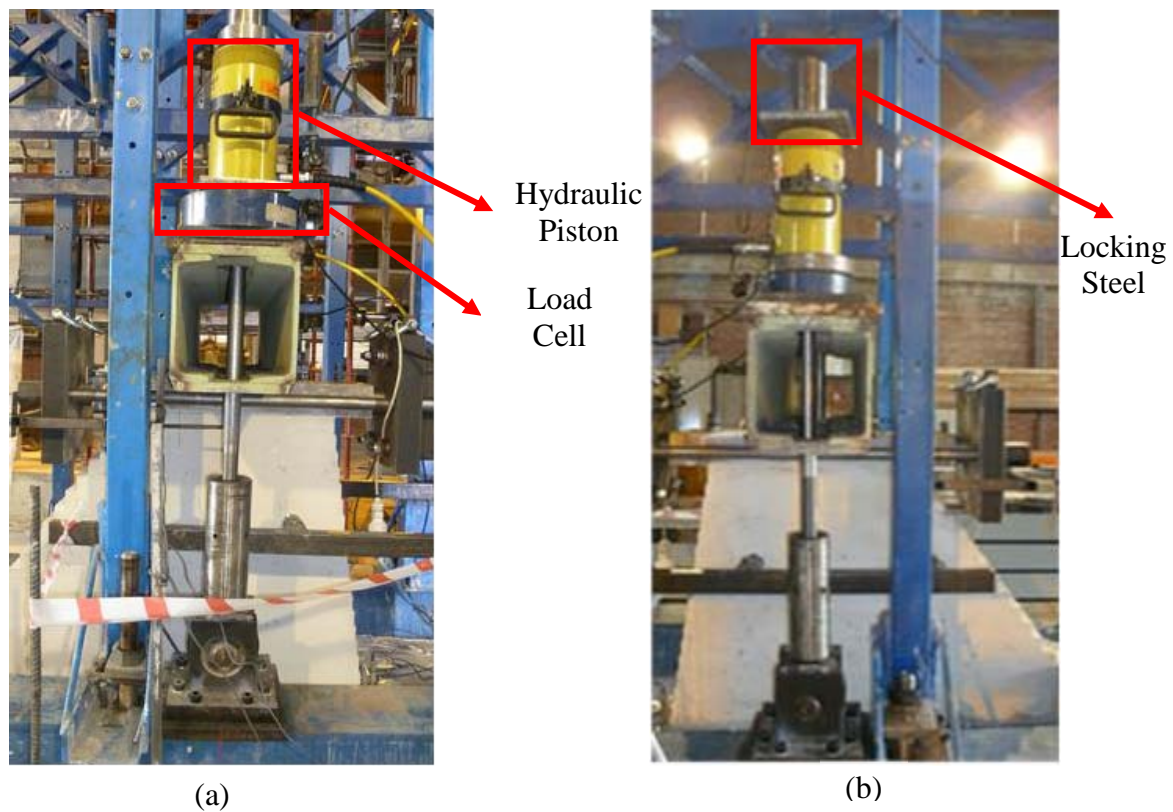Fig. 9 Dam Tip Displacement Measurements

Fig. 10 Static Axial Load Application

The experiment started with the application of a vertical load of 400 kN via tie rods (Fig. **10**) and pistons (The loading on the pistons were measured during the test and the maximum change was below 10%, which was in acceptable limits.) After that, the lateral load of 175 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 (Heidenhein) 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. During OBE, the cracks formed at the application of hydrostatic forces (Fig. 13-14) were lengthened and expanded. The maximum crack width reached 0.3 mm (Fig. 13). Besides, some capillary cracks formed between the steel loading plate and the dam body due to increased lateral load demands (Fig. 14). However, base sliding did not occur during the lowest hazard level. Therefore, this dam specimen could be claimed as well-performing under the effect of OBE.
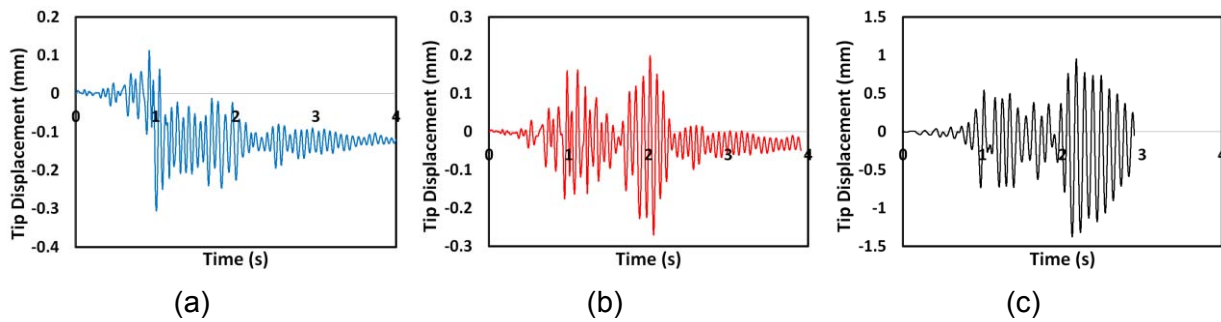
Fig. 11 Tip Displacement Demands : (a) OBE, (b) MDE and (c) MCE
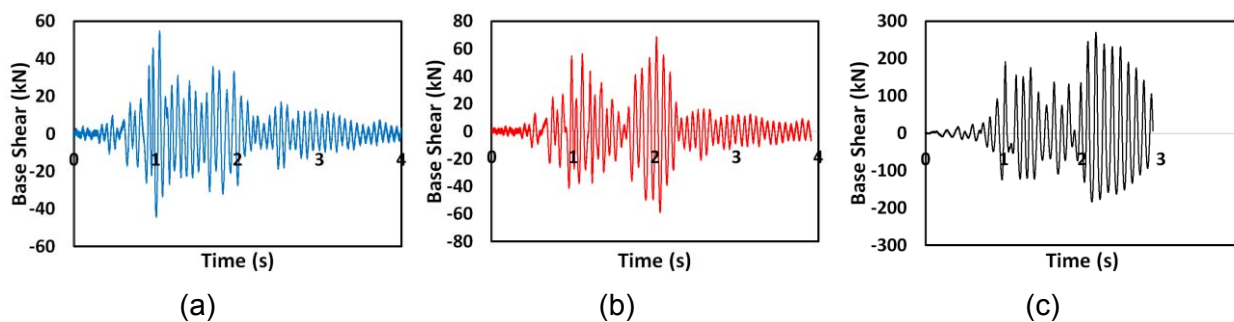

Fig. 12 Base Shear Demands : (a) OBE, (b) MDE and (c) MCE

MDE earthquake was applied to the slightly damaged specimen. From Fig.'s 11 and 12, it was clear that the load and displacement demands in MDE were about 20% more than those in OBE. During MDE, the cracks were also lengthened and expanded (Fig. 15). Some cracks reached a width of 0.4 mm. At this hazard level, a body crack (0.1 mm width and 100 mm length) was also formed (Fig. 15).

In MCE, the tip displacement demand reached 1.5 mm, which was approximately 5 times the one observed in MDE (Fig. 11). Similarly, the base shear demand was 250 kN (excluding hydrostatic effects), which was nearly 4 times the one in MDE (Fig. 12). The length of base cracks was also increased to about 1000 mm. Furthermore, the body crack, appeared in MDE, was also expanded. As the hydrostatic effects were cancelled during MCE, base cracks at the downstream face also appeared (Fig. 17). However, those cracks did not merge to the upstream face (Fig. 16-17).
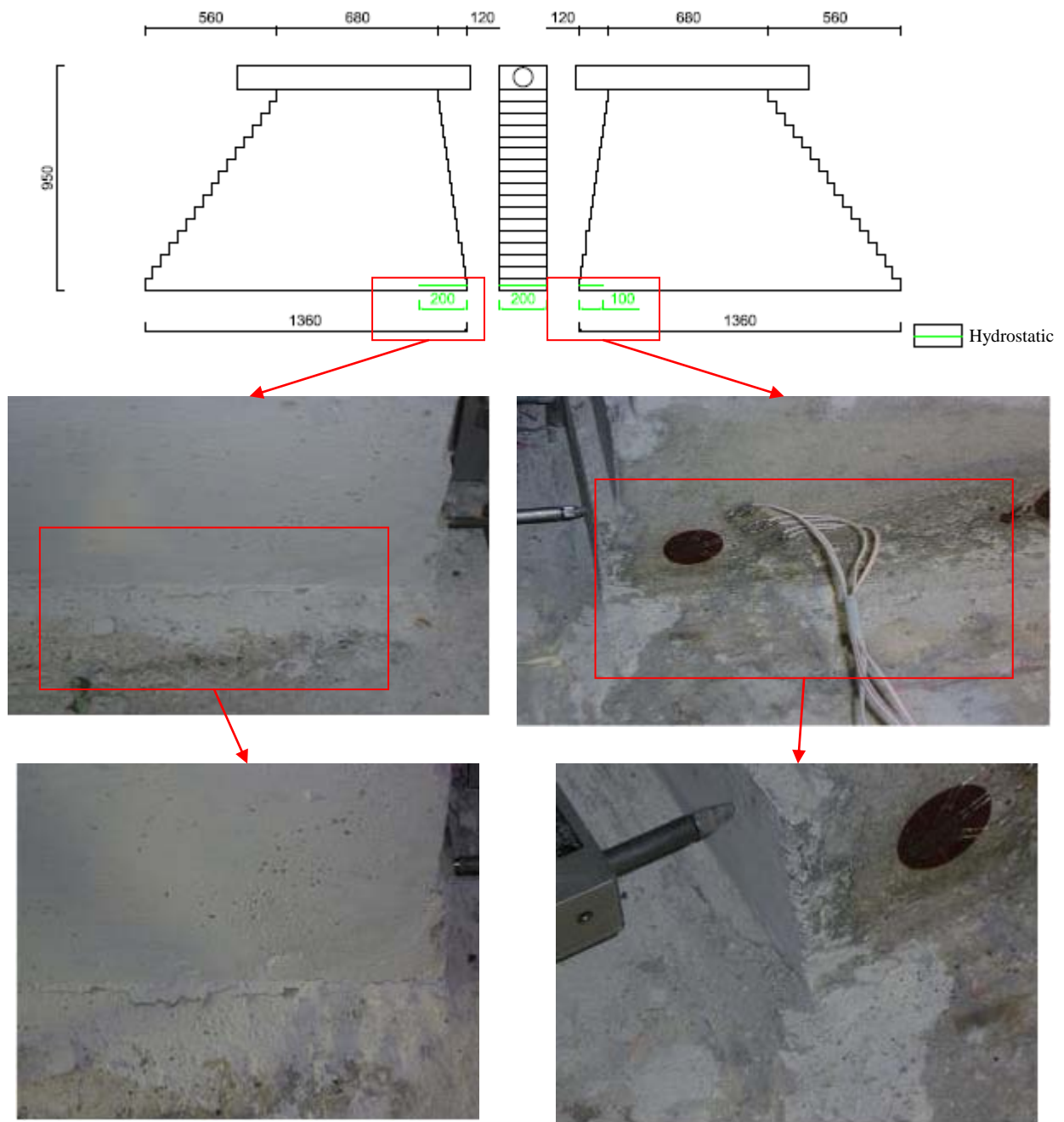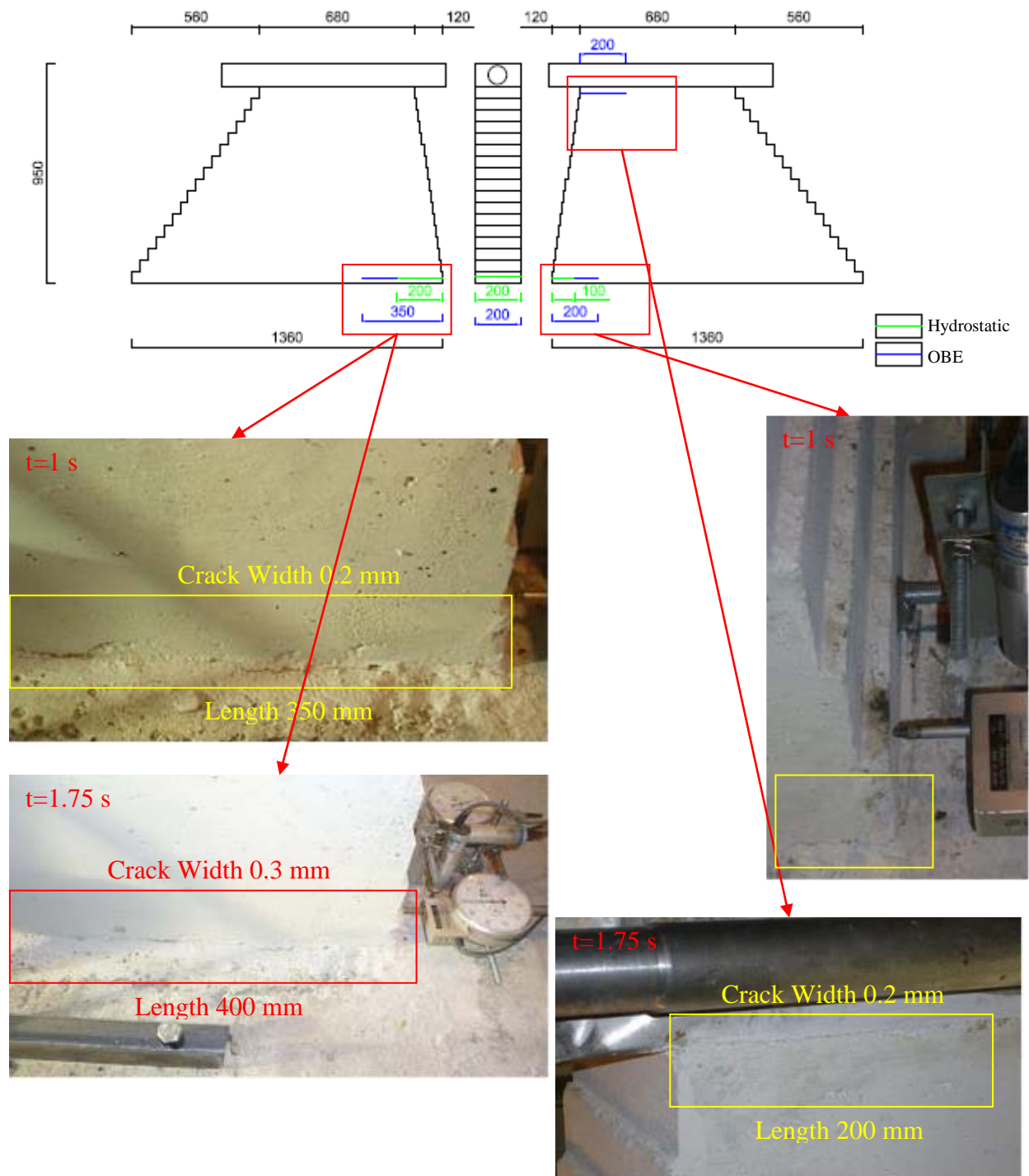
Fig. 13 Cracks after Hydrostatic Loading
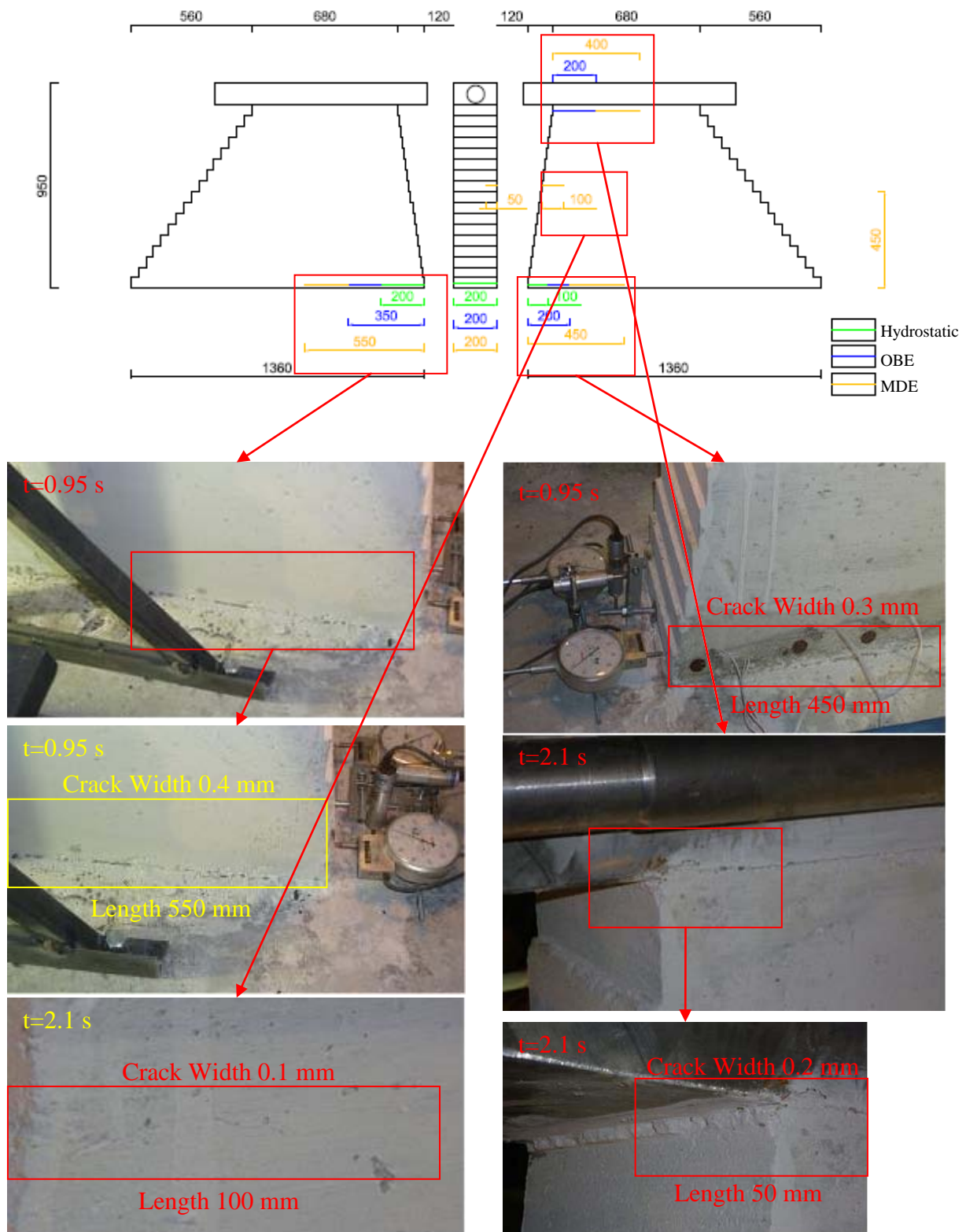
Fig. 14 Cracks after OBE
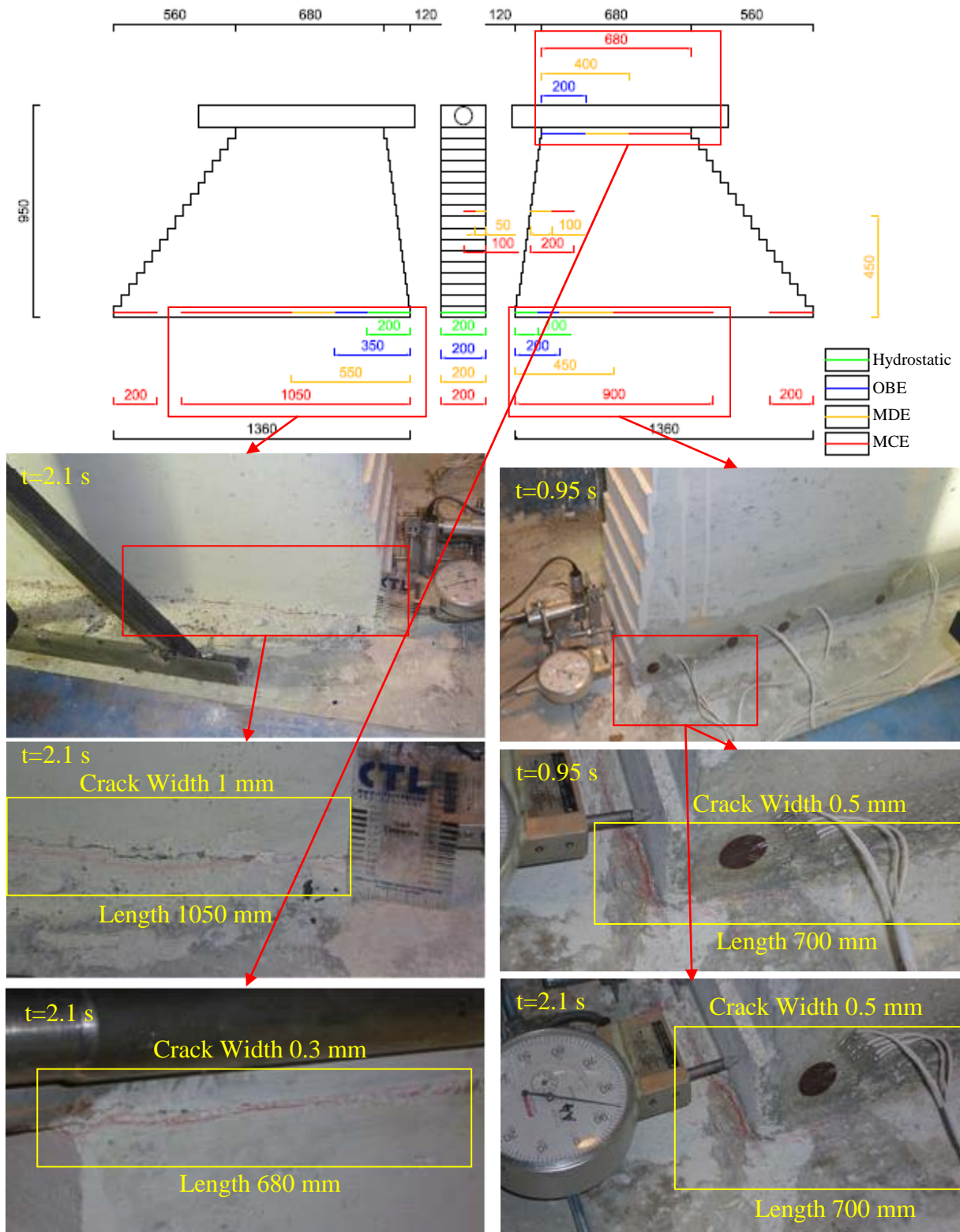
Fig. 15 Cracks after MDE
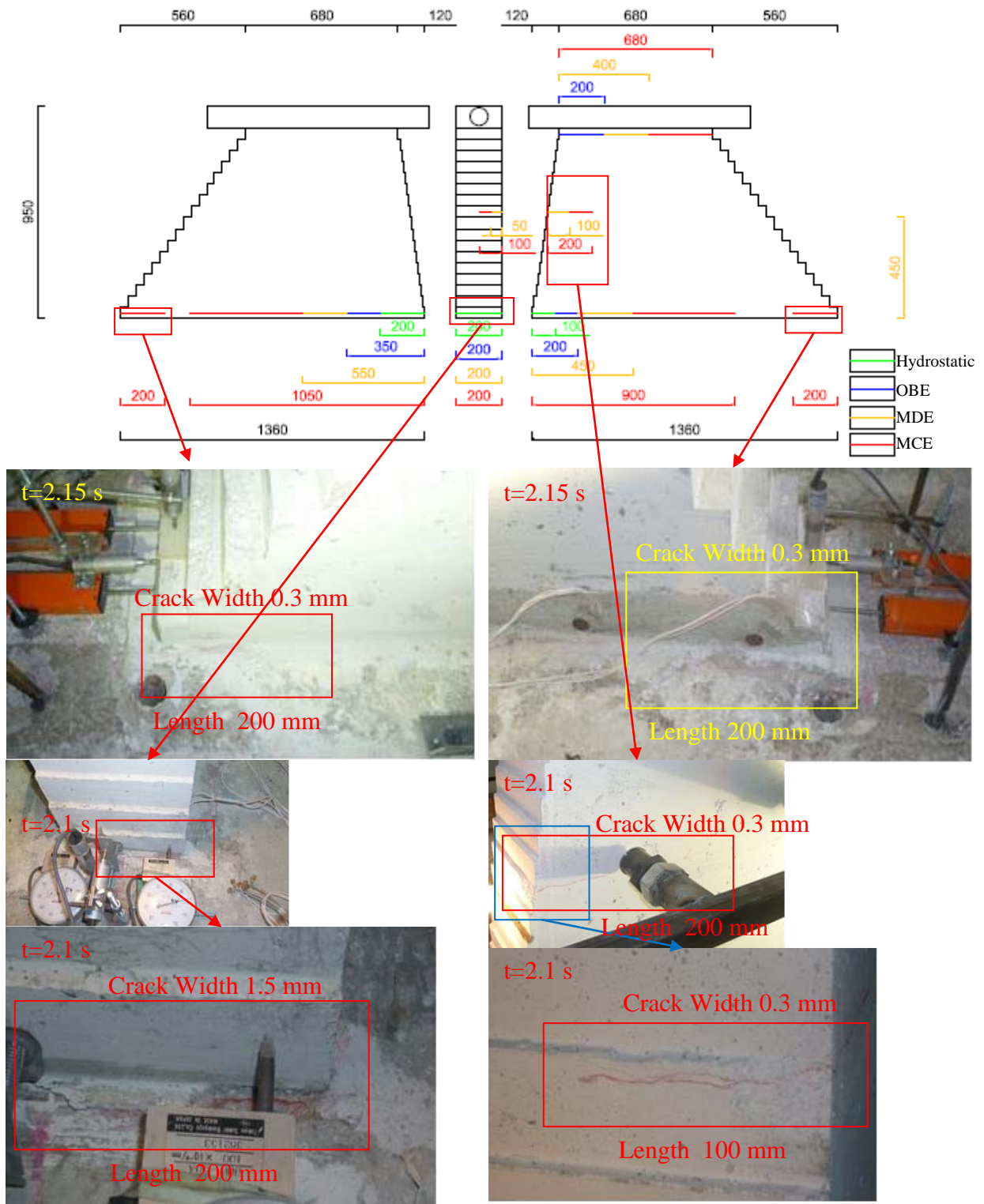
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Fig. 16 Cracks after MCE (1)

Fig. 17 Cracks after MCE (2)

5. CONCLUSIONS

Inspired from Fenves and Chopra (1984.a and 1985), a SDOF concrete gravity dam specimen was subjected to pseudo-dynamic testing in this study. Hence, the inertial forces and the hydrodynamic effects were acted on the dam specimen at one point such that the stress distributions at the base of the scaled and unscaled dams are similar during both static and dynamic loading conditions. During the experiment, three different ground motions corresponding to different hazard levels were applied, successively. At each hazard level, the displacement, force demands and damages were observed.

The first crack formed immediately after the application of static loads. Besides, the demand of OBE was quite low corresponding to 55 kN of maximum base shear demand excluding 175 kN of hydrostatic force. Therefore, the cracks formed at the application of hydrostatic forces (Fig. 13-14) were lengthened and expanded due to this earthquake scenario. Moreover, the maximum crack width reached 0.3 mm (Fig. 13) during OBE earthquake and some capillary cracks were formed between the steel loading plate and the dam body due to increased lateral load demands (Fig. 14).

In MDE, the previously formed cracks were also expanded and lengthen from upstream to downstream face as the hydrostatic effects were not beaten, again. At this point, it should be stated that the seismic characteristics of the specimen did not change much after the application of both OBE and MDE as there were only capillary cracks at the base. In addition, the damping ratio of the dam model also remained nearly constant (in bound of 3 – 4%). During MDE earthquake, a body crack showed up at the middle height of upstream face (Fig. 15).

The last earthquake scenario, MCE, managed to beat the hydrostatic effects unlike the other two scenarios. Therefore, it caused the formation of base cracks at the downstream face of the test specimen. Moreover, the base crack widths at the upstream face reached 1.5 mm. The body crack, formed in MDE earthquake, penetrated to the upstream face and reached a width of 0.3 mm. Finally, there were no base sliding and no stability problems during any of the three tests. Therefore, this dam section could be claimed as sufficient to withstand the predicted seismic source related demands.

ACKNOWLEDGEMENT

REFERENCES

Akkar, S. (2010), "Melen barajı için tasarım spektrumunun olasılık hesaplarına dayalı sismik tehlike analizi", Report No: 2010-03-03-1-01-04, METU. (in Turkish)

ANSYS Inc. (2007), "Basic analysis guide for ANSYS 11", SAS IP Inc.

Chopra, A. K. and Zhang, L. (1991), "Earthquake-induced base sliding of concrete gravity dams", J. Struct. Eng., **117**(12), 3698-3719.

Donlon, W.P. and Hall, P. (1991), "Shaking table study of concrete gravity dam monoliths", Earthq. Eng. Struct. Dyn., **20**(8), 769–786.

Dursun, B. and Gokcol, C. (2011), "The role of hydroelectric power and contribution of small hydropower plants for sustainable development in Turkey", Renew. Energy News Digest, **36**, 1227-1235.

Fenves, G. and Chopra, A. K. (1984.a), "Earthquake analysis and response of concrete gravity dams", Report No. UCB/EERC-84/10, Earthquake Engineering Research Center, University of California, Berkeley, California.

Fenves, G. and Chopra, A.K. (1984.b), "EAGD-84: A computer program for earthquake response analysis of concrete gravity dams", Report No: UCB/EERC-734, Earthquake Engineering Research Center, University of California, Berkeley, California.

Fenves, G. and Chopra, A. K. (1985), "Simplified earthquake analysis of concrete gravity dams: Separate hydrodynamic and foundation interaction effects", J. Eng. Mech., **111**, 715-735.

Ghobarah A. and Ghaemian M. (1998), "Experimental study of small scale dam models", J. Eng. Mech., **124**(11), 1241-1248.

Gogoi, I. and Maity, D. (2005), "Seismic safety of aged concrete gravity dams considering fluid-structure interaction", J. Earthquake Eng., **9**(5), 637-656.

Harris, D.W. Snorteland, N., Dolen, T. and Travers, F. (2000), "Shaking table 2D models of a concrete gravity dam", Earthq. Eng. Struct. Dyn., **29**(6), 769–787.

Mills-Bria, B., Nuss, L. and Chopra, A.K. (2008), "Current methodology at the bureau of reclamation for the nonlinear analyses of arch dams using explicit finite element techniques", The 14th World Conference on Earthquake Engineering, Beijing, China.

Rashed, A.A. and Iwan, W.D. (1984), "Hydrodynamic pressure on short-length gravity dams J. Eng. Mech., **110**(9), 1264-1283.

Turkish Earthquake Code (2007), "Specification for structures to be built in disaster areas", TEC2007, Ankara, Turkey.

Uchita, Y., Shimpo, T. and Saouma, V. (2005), "Dynamic centrifuge tests of concrete dam", Earthq. Eng. Struct. Dyn., **34**(12), 1467–1487.

Wang, J. and Chopra, A.K. (2008), "EACD-3D-2008: A computer program for three dimensional earthquake analysis of concrete dams considering spatially-varying ground motion", Report No. UCB/EERC-2008/04, Earthquake Engineering Research Center, University of California, Berkeley, California.

Westergaard, H.M. (1933), "Water pressures on dams during earthquakes", Transactions of the American Society of Civil Engineers, **98**, 418–433.