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Pseudo seismic and static stability analysis of the Torul Dam

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Abstract. Dams have a great importance on energy and irrigation. Dams must be evaluated statically and dynamically even after construction. For this purpose, Torul dam built between years 2000 and 2007 Harsit River in Gümüşhane province, Turkey, is selected as an application. The Torul dam has 137 m height and 322 GWh annual energy production capacity. Torul dam is a kind of concrete face rock fill dam (CFRD). In this study, static and pseudo seismic stability of Torul dam was investigated using finite element method. Torul dam model is constituted by numerical stress analysis named Phase2 which is based on finite element method. The dam was examined under 11 different water filling levels. Thirteenth stage of the numerical model is corresponding full reservoir condition which water filled up under crest line. Besides, pseudo static coefficients for dynamic condition applied to the dam in fourteenth stage of the model. Stability assessment of the Torul dam has been discussed according to the displacement throughout the dam body. For static and pseudo seismic cases, the displacements in the dam body have been compared. The total displacements of the dam according to its the empty state increase dramatically at the height of the water level of about 70 m and above. Compared to the pseudo-seismic analysis, the displacement of dam at the full reservoir condition is approximately two times as high as static analysis.

Keywords: displacement; concrete face rock fill dams (CFRD); static analysis; pseudo seismic analysis; seismic coefficient

1. Introduction

Water has always been one of the most fundamental necessities in human life. Drinking water, water of purposes of cleaning, agriculture, irrigation and energy have been supplied from dams in developed countries. Therefore, dams are of strategic and economic importance. However, which type of dam must be build has to be carefully investigated. There are lots of factors affecting selection of a dam type where the dam will be constructed. The main factors which affect on the decision of a dam type are geo-morphological properties of the field, where the dam will be built, fundamental and/or rock properties, distance from tectonic faults, cost of the dam and production capacity. For instance, an arch dam building needs narrow valley side and very firm soil/rock properties. In generally, a firm soil, valley and area have to be chosen suitably for dams, but fill dams can tolerate other soil types. Besides, fill dams are more economical than other dam types in terms of cost. The fill dams can be categorized clay core rock fill dam, earth fill dam, concrete face or asphalt face-rock fill dam. It can be stated that concrete face rock fill dams are the alternative for the clay core rock fill dams, because the material of clay core is not always available (Uddin 1999, Wieland and Brenner 2007).

Concrete face rock fill dam types started to be built in the middle of the 20th century (Cooke 1984). Chatsworth dam which has been known the first constructed concrete face rock fill dam was built in California (USA) in 1895. Dams which have more than 75 m height started to bring about various problems and instances of damages (sepeage, surface cracks etc.) have been observed. To solve these problems and fix the damages new construction techniques have been introduced. For instance, solving these problems of almost every dams, which were built after 1965, thin rock fill layers were constructed and vibration roller were used on each layer to compact. Using these techniques, the less settlement was measured compared to previous techniques. Horizontal joints removed, details of the upstream concrete modified, steel reinforcement located from base to crest and concrete plate thicknesses are relatively reduced. As a result, seepage and settlement problems are considerably reduced. With the improved construction techniques, these problems have been reduced to the minimum level and higher dams have began to be built. Concrete face rock fill dams are the commonly preferred type of dam in the world today because the construction of the dam is more practical, they are economical, they are safe and they do not require very strong ground/rock environment and narrow valley conditions. Visual studies are continuing to build safer dams. Deformation analysis of high CFRD considering the scaling effects were done (Raksiri *et al.* 2018). The study was conducted to evaluate the cracks on the crest of the dam body. The dam crest has a great possibility of cracking due to the uncoordinated deformation, which agrees well with the field investigation (Wei 2016). In a study on

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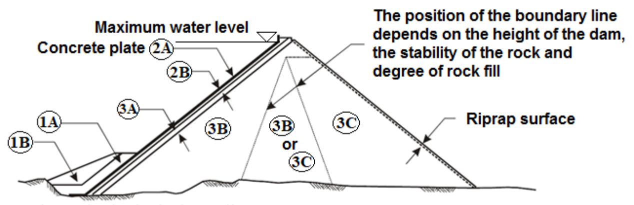

1A: Fine-grained cohesionless soil
1B: Mixed filling (silt, clay, sand and gravel)
2A: Surrounding zone filter
2B: The front face supports concrete pavement and consists of sand-gravel size material
3A: Consists of a rock filler material with a transition area characteristic of 400 mm largest grain size
3B: Consists of quarry with a maximum grain size of 1000 mm
3C: Consists of quarry with a maximum grain size of 2000 mm

Fig. 1 Typical section of concrete faced rock fill dams (Cooke 1987)

Table 1 Pseudo-static coefficients from various studies (modified from Nadi 2014)

Reference	Recommended pseudo static horizontal coefficient (k_h)	Recommended factor of safety	Earthquake effect
Seed (1979)	0.10 (M=6.5)	>1.15	<1 m displacement in earth dam
	0.15 (M=8.25)		
	0.10	>1.2	Sheffield dam (completely collapse)
	0.15	>1.3	San Fernando dam (upstream site slope defeat)
	0.15	2.0-2.5	San Fernando dam (Downstream face shifted 1.83 m (6 ft) with crest)
	0.20	1.3	Mine waste dam in Japan (dam collapse)
Hynes-Griffin and Franklin (1984)	0.5xPGA/g	>1.0	<1 m displacement in earth dam
California Division of Mines and Geology (1997)	0.15	>1.1	Unspecified
Indian Standard for Seismic Design of Earth Dams (IITK-GSDMA 2007)	0.33xZxIxS	>1.0	Unspecified
IRI Road and Railway Bridges Seismic Resistant Design Code (Marcuson 1981)	0.5xA	>1.0	Unspecified
Terzaghi (1950)	0.1 (R-F=IX)	>1.0	Unspecified
	0.2 (R-F=X)		
	0.5 (R-F>X)		
Corps of Engineering (Marcuson 1981)	0.10 (Major earthquake)	>1.0	Unspecified
	0.15 (Great earthquake)		

R-F : Rossi-Forel earthquake intensity scale
M : Earthquake magnitude
PGA : Peak Ground Acceleration
g : Acceleration of gravity
A : Ratio of design acceleration to acceleration of gravity (0.20 to 0.35)
Z : Zone factor (0.10 to 0.36)
I : Importance factor (1.0 to 2.0)
S : Site amplification factor (1.0 to 2.0)

nonlinear dynamic behaviors of earth fill dams, the importance of nonlinear behavior is emphasized in the evaluation of the stability of the earth fill dams (Terzi 2015). Another study is that optimal design of homogeneous earth dams by particle swarm optimization incorporating support vector machine approach. In this study, optimization procedure, the stability coefficient of the upstream and downstream slopes and the seepage through dam body as the hydraulic responses of homogeneous earth dam are used (Zeinab 2015). A different study by Rodriguez (2015) is to determine the geotechnical properties of earth-filled dams. This article provides a detailed description of the equipment and the test procedure, and examines a case study of its application to determine the geotechnical properties of an earth-filled dam for a tailings pond.

CFRD's are modeled and designed with appropriate software (i.e., ANSYS, PHASE and FLAC etc.) based finite element and finite difference methods to better determine stress and deformation properties of the dams. Dam safety must be ensured in all conditions. The main purpose of this study is to investigate the linear static behavior of the Torul Dam, one of the concrete face rock fill dam due to water retention at different water levels, and numerical stress and displacement analysis with different pseudo seismic coefficients. Numerical stress analysis is done by Phase2 (Rocscience 2007) program which is based on finite element analysis method as two dimensions. To investigate the effect of Torul dam body and soil, different reservoir occupancy rate and pseudo dynamic coefficients, the study was carried out in 14 different stages. A typical section of the concrete face -rock fill dams is shown in Fig. 1.

The seismic stability of soils is analyzed by the pseudo static approach, which started in the 1920s (Steven 1996), in which destructive effects are represented by fixed horizontal and/or vertical accelerations. The structural behavior of dams can be determined approximately using pseudo seismic coefficients. There are many factors that affect the proper pseudo seismic coefficient. For example, the maximum possible earthquake acceleration value or the distance to the earthquake fault line. The results of the pseudo-static analysis are directly related to the value of the horizontal seismic coefficient (k_h). The most difficult part of the pseudo-seismic stability analysis is the selection of an appropriate pseudo-static coefficient. Actually, the pseudo-static coefficients used in practice correspond to values well below peak ground accelerations (a_{max}), as the real slopes are not rigid and peak accelerations are only effective in a short time. In this regard, firstly Terzaghi (1950) proposed $k_h = 0.1$ for earthquakes with "big" (magnitude IX in Rossi-Forel scale); $k_h = 0.2$ for "severe, destructive" earthquakes (magnitude X in the Rossi-Forel scale) and $k_h = 0.5$ for "earthquakes at disaster level".

Seed (1979) made a list of pseudo-static design criteria for 14 dams in 10 countries in the earthquake zone. In 12 of them, the pseudo-static coefficients were taken between 0.10 and 0.12 with minimum safety coefficient between 1.0 and 1.5. Marcuson (1981) suggested that the appropriate pseudo-static coefficient for dams, including the magnification or shrinkage to which the dam is exposed, should be between one third and one half of the maximum amplitude. Seed and Martin (1966) and Dakoulas and

Gazetas (1986) used slip beam models and found that the inertia force on a potentially unstable slope in an earth dam is dependent on the dam response and that the mean seismic coefficient for a deep slip surface does not fall well below the crest it is very low compared to the surface. Hyness-Griffin and Franklin (1984) applied the Newark slip block analysis to more than 350 accelerograms and found that the pseudo static safety coefficient was greater than 1.0 and that “large scale” deformations did not develop in the soil dams where k_h=0.5PGA/g. As mentioned in the above explanations, there are no definite rules for the choice of pseudo-static coefficients for the design. The values of pseudo-static coefficients are found in the regulations and previous studies. Pseudo-static coefficients from various studies are summarized in Table 1.

2. Location and dam properties

The Torul Dam, which is a CFRD fill type dam, was built in Torul district of Gümüşhane province. The dam is located about 14 km northwest of the Torul town. The Torul dam constructed with the aim of energy production was completed in 2007. The annual power production capacity of the dam is 322 GW. The volume of the dam body is 4.6 hm^3 Fig. 2. The largest two-dimensional cross-section model of the Torul dam and the lake area at water level is 3.62 km^2. The thickness of the concrete plate is 0.3 m in crest and it increases by expanding towards the basin and the base is 0.7 m. The dam has a crest width of 12 m and a length of 320 m. The maximum height of the Torul dam including the crest is 142 m and the base width of the dam is 420 m.

The largest section of the dam is given two-dimensionally in Fig. 2. The views of the dam reservoir with its sides upstream and downstream directions are given in Fig. 3.

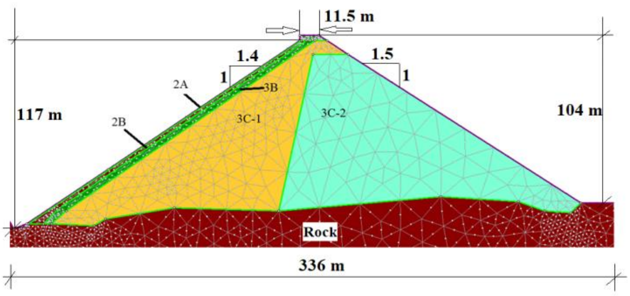

Fig. 2 The largest two-dimensional cross-section model of the Torul dam

(a) From upstream direction

(b) From downstream direction

Fig. 3 Torul dam’s views (DSI 2018)

3. Material properties

Dam body is composed of five different rock fill regions starting from the concrete plate side towards the downstream of CFRD. Five different materials were placed in the following order: Concrete plate, 2B, 3B, 3C-1, 3C-2. It is ordered from thin material to thick material.

In the dam model that was created, the rock plate and alluvial fillings were followed by rock fillings, followed by concrete slabs in the first order on the surface of the dam, and then the maximum grain diameter of 1 m in the hearth rock and the largest grain diameter of 2 m in the last downstream surface. Limestone rock mass property is defined as the Torul dam base. The physical and mechanical properties of materials used in the construction of dam and rock masses in numerical analyzes are given in Table 2. This analysis is based on the Mohr-Coulomb failure criterion and material behavior for the rock mass and dam body assumes to be elastic material behavior.

4. Boundary conditions and modelling stages

Fourteen stages were used to construct the dam finite element model (Table 3). First of all, a rock mass area (limestone) was created. The formation of initial stresses in rock mass was made numerically by gravity loading. The model is allowed to displace vertically along the vertical boundaries. Horizontal outer bottom of the model was fixed both horizontally and vertically. After the gravity loading, all displacement assigned as zero. At the second stage of modelling, all different zones of dam body (i.e., in Fig. 2) constituted into model and then gravity loading was done again. The tensile strength of the concrete used in the model is 1.6 MPa and the compressive strength is 20 MPa (Seed 1979). At this stage, the modeling of the empty part of the dam was complete. The modelling different water levels of the dam were made into stages between 3 and 13. Water level was upped to 116.65 m. A two-dimensional finite element mesh of the Torul dam and reservoir water is given in Fig. 4.

In the fourteenth stage, different seismic coefficients

Table 2 Material properties used in Torul dam finite element analysis (modified from Kartal 2010)

Material	Zone	Poisson ratio, ν	Elasticity modulus, E (MPa)	Unit weight, γ (kN/m^3)	Cohesion, c (MPa)	Angle of internal friction, ϕ (°)
Concrete	2A	0.2	28000	24.0	2.5	30
Eliminated rock and alluvial fill	2B	0.36	400	18.8	0.1	36
Rock fill	3B	0.36	300	18.7	0	50
Quarry rock-1	3C-1	0.32	250	18.5	0	45
Quarry rock-2	3C-2	0.32	200	18.5	0	40
Limestone	Rock	0.18	12060	28.4	3.8	34

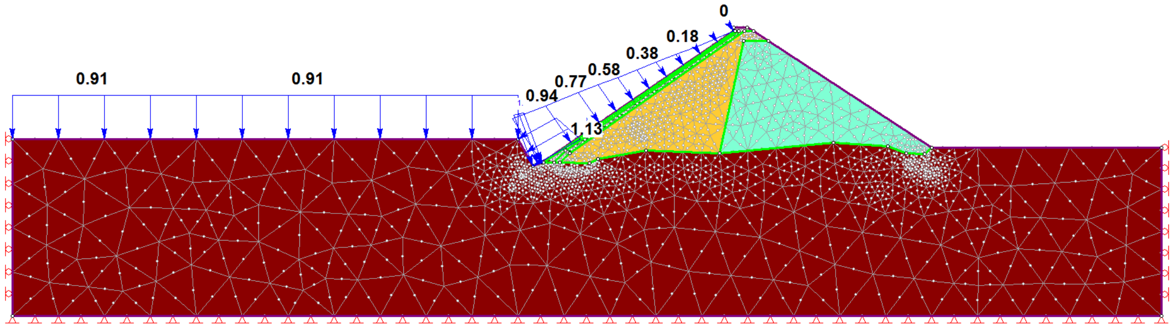

Fig. 4 Finite element mesh and water pressure on the dam is filled up to crest water level (stage XIII ; unit: MPa)

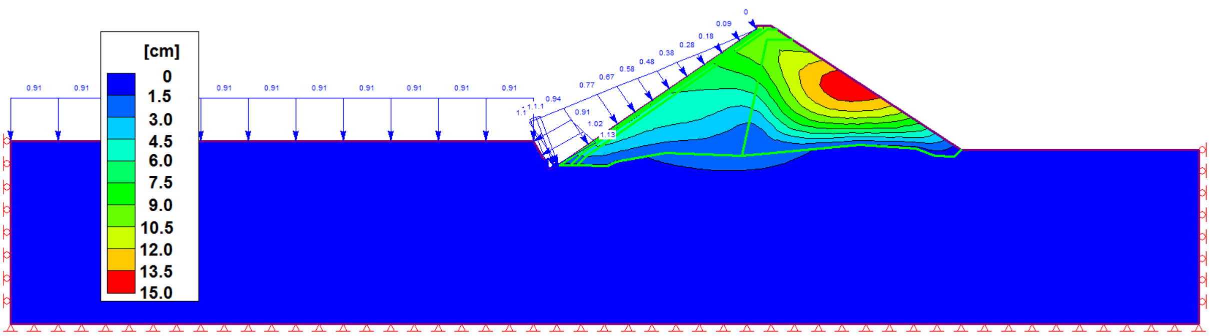

Fig. 5 Total displacement distribution along the dam body (unit: cm) (stage XIII)

Table 3 Stage description

Stage No	Explanation
I	Rock mass foundation was constituted + gravity loading
II	Dam body material was constituted + gravity loading
III	Water level: 20 m
IV	Water level: 28 m
V	Water level: 35 m
VI	Water level: 42 m
VII	Water level: 50 m
VIII	Water level: 58 m
IX	Water level: 67 m
X	Water level: 70 m
XI	Water level: 77.5 m
XII	Water level: 110 m
XIII	Water level: 116.65 m
XIV	Pseudo seismic coefficient was applied

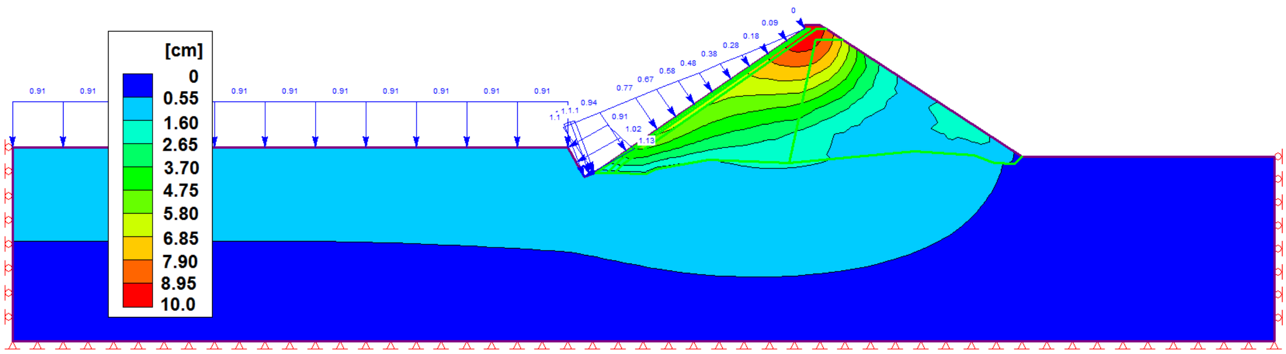

Fig. 6 Vertical displacement distribution along the dam body (unit: cm) (stage XIII)

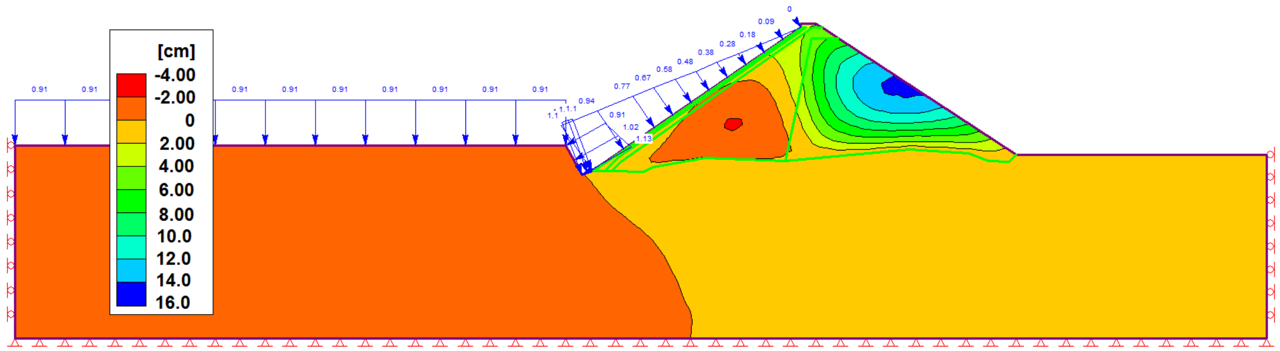

Fig. 7 Horizontal displacement distribution along the dam body (unit: cm) (stage XIII)

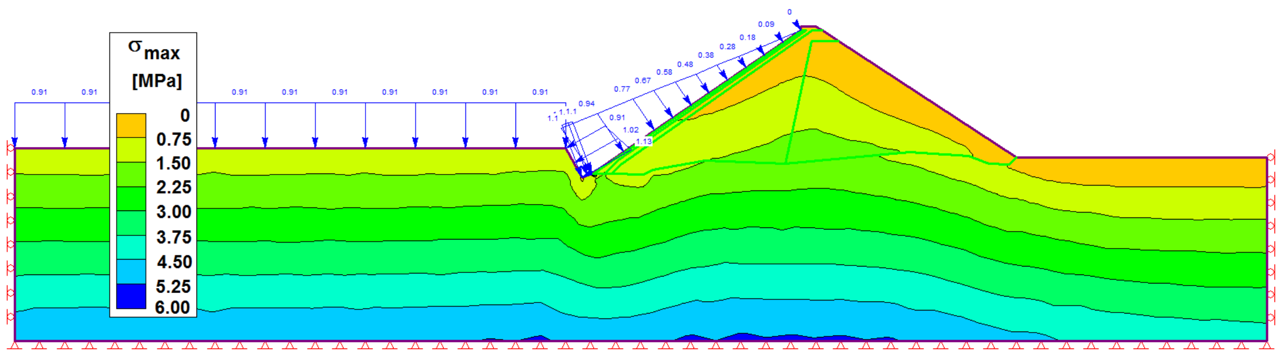

Fig. 8 The maximum principal stress distribution (unit: MPa) when the dam is full water (stage XIII)

were applied for pseudo seismic analysis. For this purpose seismic loading applied from upstream side to downstream side. The pseudo-static horizontal coefficient was chosen as a minimum of 0.1 and a maximum of 0.3. This coefficient was increased by 0.01.

5. Stability assessments

In the static analysis using the finite element analysis method, the stability of the dam was investigated at different stages under eleven different water levels. As a result of the static analysis for the case that the Torul dam was fully filled up to the crest, it was seen that the maximum displacement occurred in the middle parts of the downstream surface. This corresponds numerical modelling of the 13th stage. Stress analysis shown that the total displacement obtained as about 15 cm. Total displacement distribution is presented in Fig. 5.

The maximum vertical displacement value obtained is 10 cm in the 13th stage where the reservoir is filled up to the crest of the dam reservoir. This value was formed at the dam crest. The vertical and horizontal displacement distribution along the dam body are shown in Fig. 6 and 7 respectively.

The maximum principal stress distributions in the dam body and bedrock are given in Fig. 8. It is understood that the principal stress values are higher on the upstream side due to the water pressure.

Shear strain can be used to prediction shear failure surface on the slopes (Fan *et al.* 2015). In this study, the maximum shear strain contours was occurred on the border of the dam body and rock mass in the downstream side of dam (Fig. 9(a)-(b)). It can be said that the contours indicate a failure mechanism involving shear failure. Therefore, regions of maximum shear strain increment in the model can be considered as the critical region in regard to a possible shear failure.

The effects of water level on static stability as well as

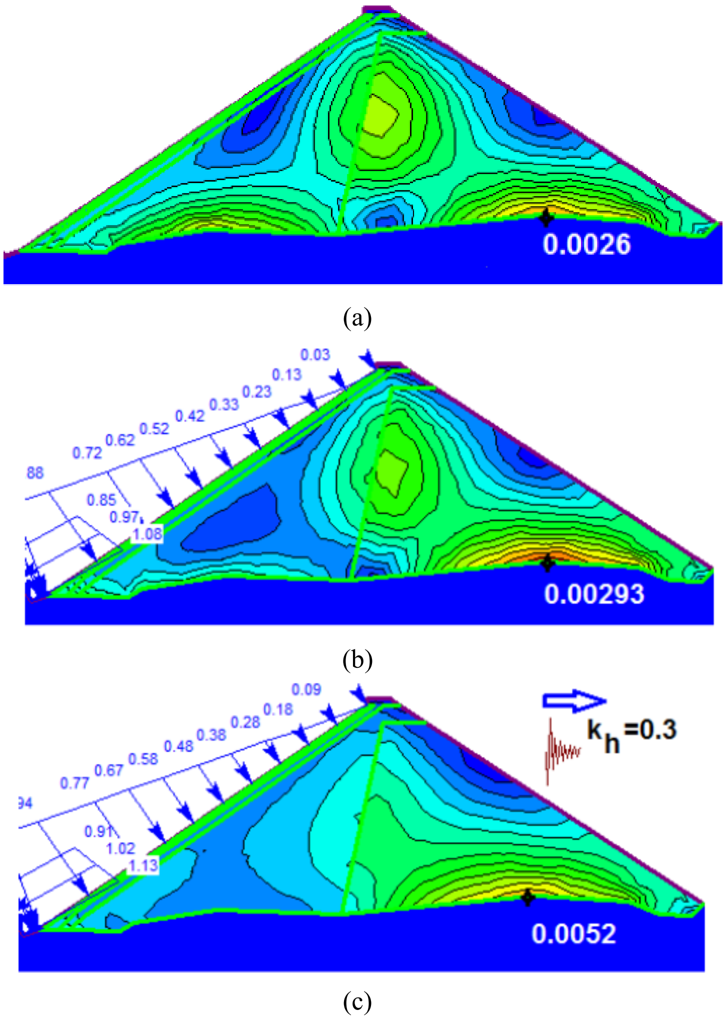

Fig. 9 Maximum shear strain (a) Stage II, (b) Stage XIII, (c) Stage XIV

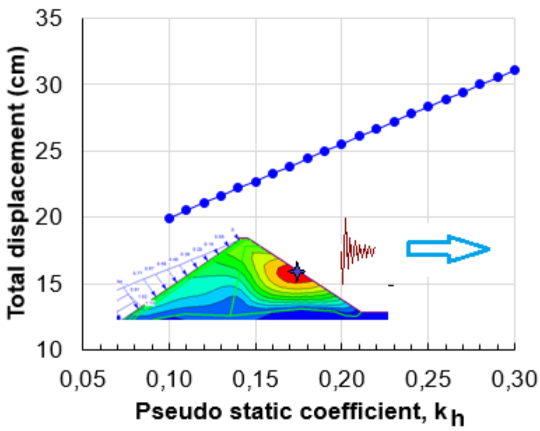

Fig. 10 The effect of pseudo-static coefficients on the total displacement

seismic movements are investigated by pseudo seismic method. In the fourteenth stage, different pseudo seismic coefficients (k_h), which was given in detail in previous section, representing the earthquake accelerations in the model were applied.

The maximum displacement value in the Torul dam body was calculated as 31.1 cm with the effect of the pseudo seismic coefficient of 0.3. It was also found that there was a linear relationship between the total displacement and the seismic coefficient (Fig. 10). Thus, as a result of the possible earthquakes that may occur in the region, the displacements in the dam are estimated. The maximum shear strain obtained in the analysis of dynamic condition for k_h=0.3 shows a 1.8 times increase compared to the static condition (Fig. 9(c)).

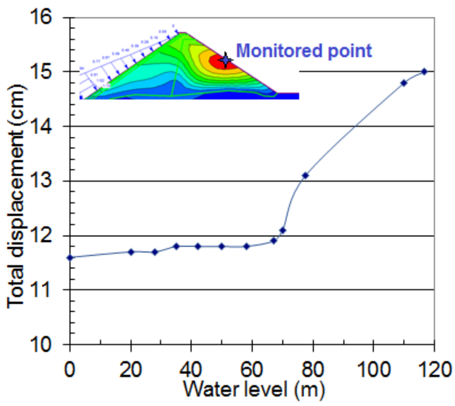

Fig. 11 Different reservoir level effect on the total displacement

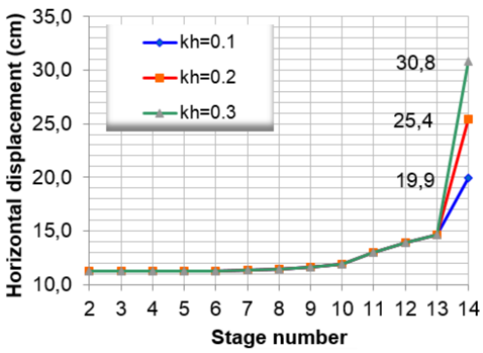

Fig. 12 Pseudo seismic analysis results for Stage 14

6. Conclusions

In this study, an example of progressive numerical stress analysis based finite element method is presented for CFRD fill dams. Firstly, analysis were performed in static condition in which the water level was changed. The changes in the water level of the dam affect the displacements in the dam body. According to the results of the total displacement obtained from the dam body, it can be seen that the displacements in the dam body showed a significant increase on the water level and above 70 m (Fig. 11). Then the pseudo-static coefficient is introduced to the model for seismic condition. The pseudo static coefficient was increased from 0.1 to 0.3 and applied only in to the fourteenth stage of the model. When the seismic condition is compared, the displacement occurring in the dam in the case of the filled reservoir is about two times the static condition ($k_h = 0.3$) (Fig. 12). It is evident that the pseudo seismic coefficients affect displacement of the dam body considerably. Therefore, it is very important to select the correct pseudo seismic coefficient in the design of dams.

However, it is very important to monitor the size and the vectors of the displacements that will occur in the dam body during the operation period of the dam. The comparison of the displacement values obtained by numerical analysis with the actual values formed in the dam will enlighten on the researches to be carried out. The validity of some approaches assumed in material behavior and properties determination with back analysis will also be tested.

Especially since most of Turkey is located in the seismic zone in terms of earthquake, the behavior under dynamic loads as well as the static conditions in the design of the dams to be constructed should also be examined. Dynamic loads are an important necessity for evaluating new stress distributions and displacement quantities to be formed in the resultant dam body by a possible pre-earthquake numerical analysis. The results presented in this study are valid for the idealization of material behavior and also need to be compared with field observations

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