Obtaining a Real Natural Numerical Landslide Model by Comparing Measurement and Predicted Values: A Case Study of Kavşakbendi Dam Left Bank Andirap Landslide

Resul Pamuk (resul.pamuk@enerjisauretim.com)
Yıldız Teknik Üniversitesi: Yildiz Teknik Universitesi https://orcid.org/0000-0002-4033-0675

Murat Eröz
Istanbul Teknik Üniversitesi: Istanbul Teknik Universitesi

Kemal Tuncer
Istanbul Teknik Üniversitesi: Istanbul Teknik Universitesi

Havvanur Kılıç
Yildiz Technical University: Yildiz Teknik Universitesi

Research Article

Keywords: Numerical analysis, monitoring, landslide mass, geodetic and inclinometer devices

Posted Date: August 2nd, 2021

DOI: https://doi.org/10.21203/rs.3.rs-687596/v1

OBTAINING A REAL NATURAL NUMERICAL LANDSLIDE MODEL BY COMPARING MEASUREMENT AND PREDICTED VALUES: A CASE STUDY OF KAVŞAKBENDİ DAM LEFT BANK ANDIRAP LANDSLIDE

Murat Eröz[1,2], Resul Pamuk[2], Kemal Tuncer [2], Havvanur Kılıç[3]

Abstract

The Andırap landslide is located on the left bank of the Kavşakbendi dam approximately 50 km from the Kozan district of Adana province in Turkey. After it was determined that the mass stability was impaired during the dam construction work, the landslide movements were followed by surface geodetic measurements and inclinometers. According to the measurements that were taken, displacements totaling 0.10 m were measured between 2010 and 2017, and it was determined that the speed of movement slowed considerably between 2017 and 2020. In this study, the results of stress-strain and stability analyses were evaluated taking into account the soil model created based on extensive site and laboratory research to examine the sliding mechanism of the Andırap landslide mass. After the numerical model was verified using site measurements, the movements of the landslide mass were examined by numerical analyses, taking into account the different loading conditions that may be encountered during the service life of the dam. According to the results of the analysis, no global slide was observed for the slip circle of the Andırap landslide and in the analyses conducted for the situation where the reservoir is full, the deep displacement of 0.11 m was consistent with the average displacement of 0.04 and 0.11 m deformation values measured from inclinometers. In the analyses carried out for the loading condition featuring a full reservoir and earthquake effects, it was calculated that shallow displacements reached up to 1.0 m, but deep displacements were 0.13 m.

Keywords: Numerical analysis, monitoring, landslide mass, geodetic and inclinometer devices

1 Enerjisa Enerji Uretim A.S., Barbaros Mh Çigdem Sk Agaoğlu My Office İs Merkezi No: 1/15b–15c-16, 34746 Atasehir / Istanbul, Turkey, murat.eroz@enerjisauretim.com (M.E.); resul.pamuk@enerjisauretim.com (R.P.); kemal.tuncer@enerjisauretim.com (K.T.)

2 Enerjisa Enerji Uretim A.S., Barbaros Mh Çigdem Sk Agaoğlu My Office İs Merkezi No: 1/15b–15c-16, 34746 Atasehir / Istanbul, Turkey

3 Department of Geotechnical Engineering, Civil Engineering Faculty, Yildiz Technical University, Esenler, Istanbul 34220, Turkey; kilic@yildiz.edu.tr

Introduction

To reduce the problems and risks encountered during the design and operation of structures with long service life, such as dams, the properties of each case should be considered, including regional geology, topographic characteristics, presence of water resources, static and dynamic loading conditions, etc. Along with these conditions, which can vary a lot regionally, the condition of the surrounding slopes due to the construction of the structure must also be considered in the design. In geotechnical designs, a design is usually performed after the current state of the land is fully determined, taking into account the possible loading situations that the structure will be exposed to during its service life. Numerical analysis can be done by taking into account the site model created as a result of realistically determining the underground soil sections, also factoring in the studies carried out in the site and the laboratory. The success of modeling and numerical analysis is very related to the precise determination of the general geology, structural geology and stratigraphy of sliding slope. The geology of the slope can usually be determined by drilling in underground soil sections that will cover places that best represent the sliding mass. Material parameters determined by insitu site tests conducted on-site during drilling and laboratory tests conducted on soil samples taken are used in numerical analyses. By determining correct loading and drainage conditions and material parameters of the site model and selecting appropriate analysis methods for the stress-strain analysis, potential displacements can be predicted with a high degree of accuracy. In the design and performance evaluation of dams, it is the most accurate approach to determine the soil profile in the site and transfer the conditions of the site to numerical analysis. The numerical model established as a result of the studies is validated once it's confirmed that analysis results are consistent with monitoring measurement results. Thus, ground movements can be realistically predicted, according to deformation analyses, taking into account the possible loading situations that the structure will be exposed to during its service life.

In Li et al. (2016), it was emphasized that numerical analyses to assess the stability of slopes are often not sufficient on their own, but must necessarily be supported by monitoring results. In Zhou et al. (2016), the stability of the 530 m-high left coast excavations in the Jinping-I hydropower project was examined and the complex geological structure was shown as the reason why the left bank excavation was very critical. A very detailed monitoring system was installed to monitor deformations, and measurements were made with both surface deformations and extensometers. On the other hand, by modeling the slope and excavation with a 3-dimensional numerical analysis, the development of deformations due to slope excavation was analyzed and the calculation results showed that the sliding depth was consistent with the measurements. As part of the analysis of the Wujiang landslide with a volume of 1,327 × 107 m^3 in Sun et al. (2016), geotechnical characteristics and formation properties of the landslide mass were studied. Taking into account the determined parameters, the impact on the Key Water dam, which was built 300 m to 590 m away from the landslide mass, was investigated by numerical analysis. As a result of the study, it was estimated that the shift occurred in the shear zone containing cracks in the bedrock, and although this landslide is an old landslide mass, it is currently evaluated as a repeated landslide mass. In further deformation analyses, it was determined that the landslide was still in the process of slow creep deformation. Thus, it was noted that the upper part of the landslide is likely to slide due to a superficial fracture, and that the probability of a global slide is low. As a result of stability analysis using the Morgenstern-Price method, it was stated that the decrease in strength parameters of the soil negatively affected stability, especially due to the rise of the reservoir water level. On the other hand, after this reduction, it was stated that the reservoir water increased stability by applying pressure to the dam fill surface. However, it was emphasized that the landslide should be monitored during and after the construction of the dam beacause the factor of safety calculated in the areas close to the dam body is quite low. To examine the stability problems that occurred during the slope excavations in Çitlakkale region within the borders of Giresun province of Turkey, in the study conducted by Kaya, (2017), detailed geotechnical studies were carried out to determine the failure mechanism, and measures were taken to increase the stability of the slope. Similarly, inclinometer measurements were carried out, the motion speed of the landslide mass was determined, and the stability of the slope was examined using limit equilibrium and shear strength reduction methods. It has been stated that stability problems in the slope are caused by excavations, and a support wall must be built on the heel to prevent the slope from sliding. In Bednarczyk et al. (2019), measurements taken with monitoring instruments placed at numerous mine sites in Poland found displacement movements at a depth of 235 m, and this finding helped the employer to take less risk during excavation. It was emphasized that displacement and pore water pressure changes are also important in terms of using them as an early warning indicator, and it was important that the detailed numerical analysis that was carried out should be verified taking into account local site conditions and possible hazards. In the study to investigate the root causes of the landslide occurring in the Maharashtra region of India, the landslide mass was modeled by taking into account the material parameters determined from soil samples taken from the site. It was found that the cause of the slide was a negative impact on the strength of the soil as a result of excessive rain. In Shah et al. 2019, site use maps showing the cause of sliding were examined using Arc GIS monitoring systems and excessive pore water pressure elevation zones were identified that caused the slope to shift. In the study, agricultural effects caused by drainage systems and excessive pore water pressure rise in the development of the landslide mechanism were mapped and monitored, and recommendations were made to increase stability by controlling the factors causing the slide. In the Coltorti et al. (2019) study, which focused on the activities and characteristic features of complex landslide structures in the Tuscany region of Italy, it was stated that lithology and downstream river valleys may be the main cause of movement, especially due to gravitational influence, and to confirm this activity, the study performed multi-temporal analyses with the Orthophoto system in 4 separate periods: 1954, 1988, 1996 and 2013. It was emphasized that the activity classifications obtained as a result of the study are also applicable to other landslide areas with similar zones. In Wang et al. 2020 study, the stability of the Duanjiagou landslide was examined by analyses using the force transmission

method. The force transmission method is a method that is mainly based on the limit equilibrium analysis, and provides solutions based on the principle of force balance divided into a certain number of ground columns of floating ground mass. The strength parameters of soil samples taken from landslide material in saturated and residual conditions were obtained from direct shear tests. It was stated that the results determined using the force transmission method and the results obtained from numerical analyses are consistent with each other.

The Andırap landslide is located on the left bank of the Kavşakbendi dam (*Fig 1*), about 50 km from the Kozan district of Adana province in southern Turkey. Following land work in 2009, the construction of the dam was completed between 2012 and 2013 with the completion of the dam fill and the face slab concrete pavement works in late 2013. Although the mechanism of the landslide was first defined in the 1960s, it was noticed in site observations made in 2009, whereby surface movements were followed by geodetic measurements and deep movements by inclinometers, and such observations continue to be made. Depending on the movement of the landslide, possible risks were evaluated both during construction excavations and after the dam was commissioned. In this context, situations that would halt the progress of the project and might cause the dam body to fail due to mass slides within the reservoir during the operation phase were expected to be taken into account. To that end, extensive research and measurements have been carried out on the landslide to understand and prevent its mechanism. Geological and geomechanical characteristics of the survey area were examined in the Geoconsult (2009) report, and geological units and discontinuities were identified in the region. Geological, tectonic and stratigraphic properties of the region at large and the specific tectonic and structural features of the area including the Kavşakbendi dam were examined by Özgül et al. (2002). Engineering geology properties of the region and groundwater condition were investigated by Yüzer, (1971) and Ulusay et al. (2008), and the stability of the landslide mass was evaluated by performing stability analyses under different loading conditions. In a study conducted to investigate landslide movements that occurred after the excavation of the dam began, the geological layers of the landslide were examined by taking into account the core drillings that were carried out. After obtaining detailed geological data, strength parameters were determined by laboratory tests and the stability of the landslide was investigated by the limit equilibrium analysis and necessary measures were taken. In this context, to reduce the forces in the sliding direction above the landslide mass, the mass was pallied by digging on the slope and drainage trenches were opened on the excavation pallies to prevent surface water from leaking into the slope, allowing the water to be removed in the fastest manner possible. Also, inclinometers and geodesic surface deformation gauges were installed to monitor deep and surface deformations that were not visible during the excavation of the dam body (Jung and Verdianz, 2013). Following studies conducted in 2013, both deep landslide movements and surface geodetic measurements and landslide movements continue to be monitored until today. On the other hand, surface deformations are monitored annually using precision air tools to increase the accuracy of surface measurements. Besides, in cases where an increase in the speed of movement has been observed in the period since 2014, drainage trenches have been excavated to reduce the impact of surface waters on the slope, and existing drainage trenches have been cleaned. To compare the measurements and deformation results, stress-strain analyses and limit equilibrium analyses were carried out in different loading conditions starting from 2018, and slope behavior in the loading conditions that the landslide may encounter in the future was investigated.

In this article; we present site, laboratory and numerical studies conducted as part of the determination and prevention of the deep sliding movement called the Andırap landslide that occurred on the left bank of the Kavşakbendi dam. The Andırap deep sliding movement occurred in the main contact of limestone and shale, which is approximately 72 m deep from the ground surface. After it was predicted that this landslide would create a risk to the structure of the dam, the topographic structure of the site and geological formations were studied in great detail to investigate the behavior of the landslide mass, and the exact location of the sliding movement was determined. In the area where the landslide occurred, the sections of the underground geological units were examined in great detail and an as-built 3-D numerical model was created for numerical analyses. After the numerical model created using the stress-strain analysis of two critical sections was validated, the possible shear movements of the slope were investigated using stress-strain and limit equilibrium analyses taking into account the potential loading conditions that might impact the stability of the dam throughout its service life.

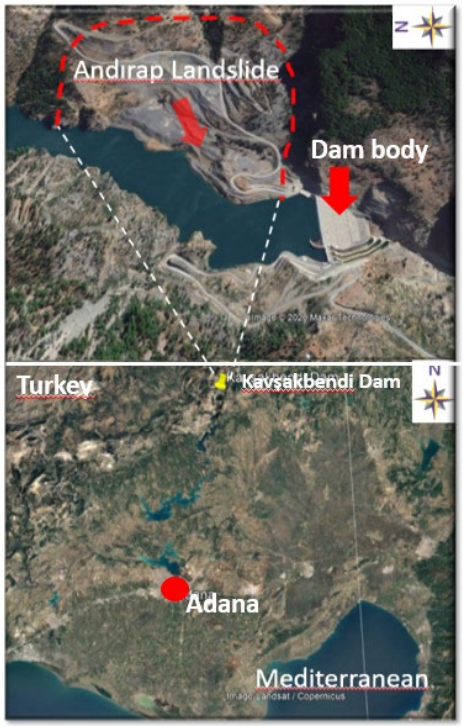

***Fig 1**. Kavşakbendi dam and the location of Andırap landslide (http://www.map.google.com)*

METHODOLOGY

General geological properties, stratigraphy and structural geological properties of the site were investigated in detail and a general geological map was made to determine the movement mechanism of the sliding mass of the Andırap landslide. For this purpose, a general geological map, geological structural stratigraphic units and formation thicknesses of geological units showing sliding behavior were determined. Later, core drilling was carried out at planned locations in the landslide zone and site tests were carried out to determine the actual thickness of the geological units causing the slide and the engineering parameters on the obtained core samples. In the area where the landslide occurred, a numerical model was created for two critical sections and stress-strain analyses were performed. By comparing the results of the analysis with 10-year monitoring measurements, the numerical model established to study site soil behavior was validated. Taking into account this numerical model, the behavior of the landslide mass was studied for 3 different loading conditions. In this context, loading conditions were taken into account that includes the empty reservoir state (LC-1), the full reservoir state (LC-2) and both the full reservoir state and the effects of earthquakes (LC-3) (Akkar et al. 2009). In this study, the behavior of the slope in case of water drawdown from the dam reservoir was not studied.

In numerical analyses, the Mohr-Coulomb failure hypothesis (Coulomb 1776), presented in equation 1 was used as the criterion of failure on the sliding surface. Strength parameters for short-and long-term stability analyses were determined by consolidated-undrained (CU) and consolidated-drained (CD) triaxial tests. Strength parameters for shale and limestone texture in the zone where the slide occurred were determined from CD tests for drainage conditions (Jung and Leitner, 2011).

$$\tau = \sigma'.tan\varphi' + c' \qquad \text{(equation 1)}$$

In this equation, τ is the shear strength of the soil, c' effective cohesion, σ' the normal effective stress acting on the shear surface, and φ' the effective shear strength angle.

Simplified Bishop and Spencer methods were employed in the Slide 6.0 program for stability analysis using the limit equilibrium method (Abramson et al. 2002) based on a circular sliding surface. The Diana FX10.1 program, which provides solutions via the finite element method, was used in the stress-strain analyses. In 2-dimensional stability analyses with Diana FX 10.1, the factor of safety against failure was determined by strength reduction (Dawson et al. 1999; Hammah et al. 2004). In this method (equation 2), the cohesion and friction component in the Mohr-Coulomb failure criterion is gradually reduced (by dividing by the increased factor of safety (F)). The reduction of strength is maintained until the shear stress is equal to the shear strength; that is, until there is a failure. The factor of safety at the moment of failure is indicated as the factor of safety against sliding of the slope being studied.

$$\tau = \frac{\sigma'.tan\varphi'}{F} + \frac{c'}{F} \qquad \text{(equation 2)}$$

While creating the site model primarily for numerical analysis, in the AutoCAD Civil 3D® program, sections of the area were obtained by matching the topographic contour maps of the area and imported into the programs by saving them as .dxf files, having processed the geological units acquired from the drilling logs. Later, an analysis was carried out by taking into account the soil structure relationships, analysis method, material parameters and loading conditions. Standards applied in laboratory tests on soil and rocks to determine the material engineering parameters used in the analyses are also presented in Table 1. During the drilling works, a pressuremeter tests were conducted at 1.5 m intervals following TS EN ISO 22476-4, ISO 22476-4 and ASTM D-4719-07 test standards.

Table 1 Laboratory tests and applied standards

	Tests	**ASTM**	**Turkish standard**
Soil	Classification according to USCS	ASTM D2487	
	Grain size distribution	ASTM D 422	TS 1500 / TS 1900-1 / 2
	Hydrometer analysis	ASTM D 422	TS 1900-1
	Atterberg limits	ASTM D 4318	TS 1900-1
	Moisture content	ASTM D 2216	
	Specific gravity & absorption	ASTM C 127	
	Permeability	ASTM D 5084	
	Specific gravity test	ASTM D 854	
	Consolidation / oedometer test	ASTM D 2435	
	Relative density	ASTM D 4254	
	Ring shear test CD	ASTM D 6467-06a	
	Triaxial test (Fr/CD)	ASTM D 4767	
	Cyclic triaxial test	ASTM D 5311	
Rock	Rock sampling	ASTM D 2113	
	Basic friction angle	ASTM D 5607	
	Direct shear test on a saw-cut surface of rock species		
	Uniaxial compressive strength	ASTM D 2938	
	Elastic modulus of rock sample	ASTM D 3148	

ANDIRAP LANDSLIDE AREA AND SOIL PROPERTIES

A map showing the general geology of the Andırap region is presented in *Fig 2* and the geological units on the right and left slopes of the dam body are shown. The slopewash forms the top layer of the Andırap region and its thickness varies from approximately 5 m to 30 m (Jung and Verdianz, 2013). Block and rock-fragment flow sections represent a younger geological unit than slopewash flows and the fossil landslide, and were formed as a result of material flows from massive limestone masses during periods of freezing dissolution. Especially block falls and flows were found in the sections where the Andırap region and massive limestones converge. Block flows are not an element that will trigger a global slide in the region in any shape or form, but is one of the geological components. The parts shown in light yellow in *Fig 2* are slopewash and show sediments, especially in the upper parts of the Andırap landslide zone. Geological units shown in a very light brown are limestone units and are one of the main geological units within the stratigraphic structure of the landslide mass. Limestones are a mass of rock that in general resist the sliding movement of landslides, display karstic and weathered in places, but are classified in the medium-good rock mass class. The shale is found stratigraphically under limestone rocks in the study area, and there are areas where it rarely surfaces. These units are designated in magenta-purple on the map (Jung and Verdianz, 2013). Liquid limit and plastic limit values for the clay soil were determined on average as 29% and 17%, respectively. Also, the unit volume weight of the clay soil was identified as 22.5 kN/m^3 and the cohesion value as 15 kN/m^2 (Jung and Verdianz, 2012). Geological units located on the right and left banks of the dam structure are shown in *Fig 2* (Jung and Verdianz, 2013). Within the scope of this article, the mechanism of the deformation-shear movement occurring in the fabric of limestone and shale geological units was taken into account and the behavior of slope mass was examined by numerical analysis. In *Fig 2* the view of the slopewash units surfaced in the top layer of the research area is presented.

In the Andırap region, limestone is one of the bedrock masses in the dam body and upstream cofferdam (*Fig 2*). In this formation, karstic cavity zones, stuck clay masses, quartzite and sandstone transitions and dolomitic limestone levels are

observed in places. Dip directions of layers are generally from the mountain to the dam body. Layers include intermediate clay layers and sliding planes.

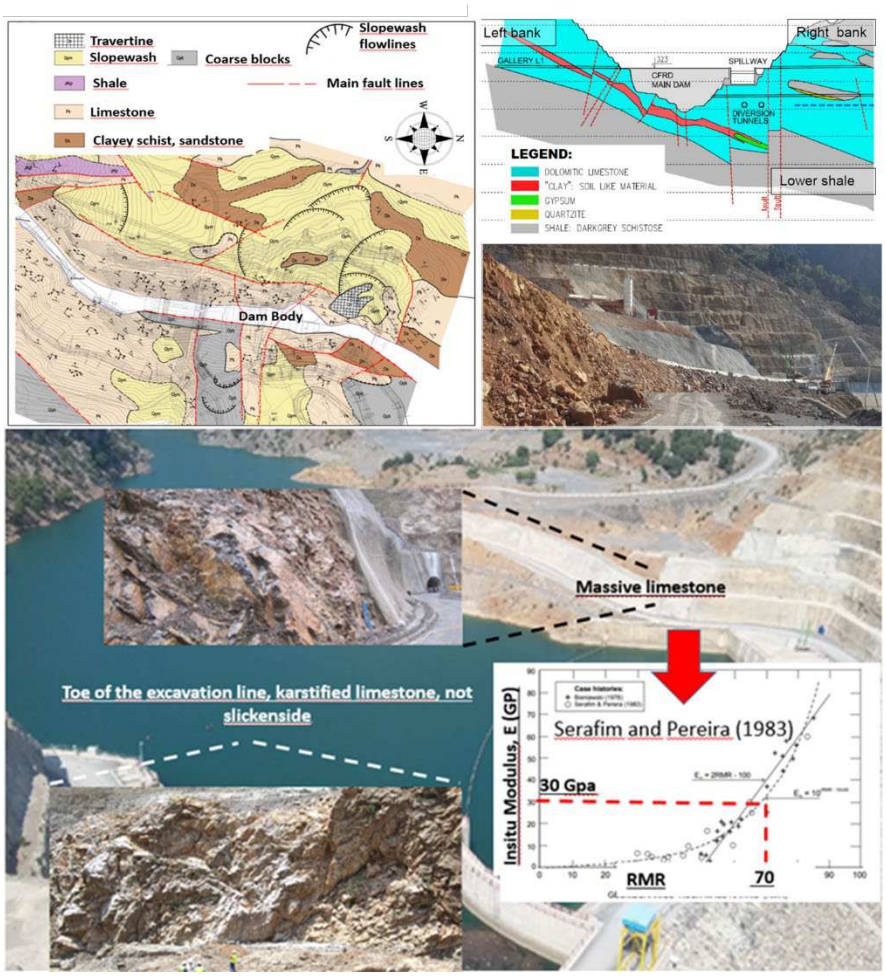

Fig 2 General geology map, geological section and general view of slopewash material on the top & positions of bedrock limestone mass in the dam body

As part of the article, an as-built 3-dimensional numerical model has been created for the region to create a realistic numerical model by precisely determining the current state of the site. For this purpose, a photogrammetric map was obtained with 80% overlay aerial photos taken from 150 m flight altitude with a Multirotor G4 surveying robot unmanned aerial vehicle. Numerical maps were created by producing a point cloud with an accuracy of 0.04 m GSD from photos processed using Pix4D photogrammetry software. 3-D maps obtained from these measurements show the topographic state before and after the reservoir is filled in *Fig 3*. A topographic surface map of the land was obtained with precise flights made in the region. Thus, the sections (Section 1-1 and Section 2-2) where stability analyses were performed were created to best represent the site topography. Using these maps, a topographic contour map was obtained by comparing old and new topographic maps in AutoCAD Civil 3D (*Fig 3*). Using this topographic map, critical sections to be analyzed were obtained using the AutoCAD Civil 3D program. In *Fig 3* the parts indicated in gray represent the map created in 2010 and the coordinates are combined with the topographic map created in 2017 and shown in pink. Thus, an up-to-date surface map of the Andırap landslide has been obtained, ensuring that the sections best represent the underground soil structure. After obtaining topographic maps, drilling points were placed together with their coordinates on the contour map of the landslide and sections 1-1 and 2-2, where a numerical analysis was performed, are shown in *Fig 3* in order to process geological units in the mapped sections.

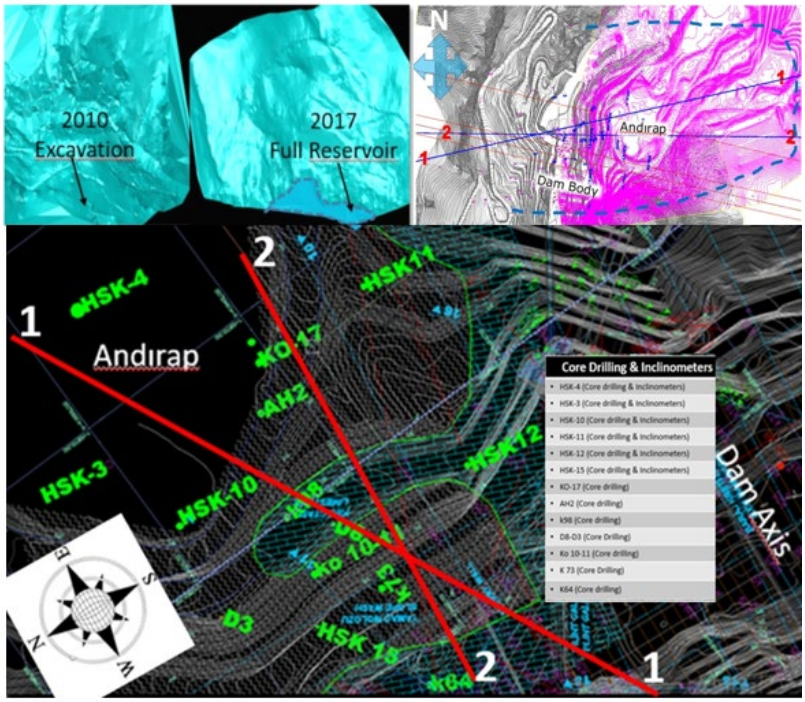

Fig 3. 3D topographical maps (turquoise), impoting into AutoCAD Civil 3D (gray and pink contour lines), determining the 1-1 and 2-2 sections

In sections 1-1 and Section 2-2, where a numerical analysis was performed, displacements from inclinometer and topographic surface readings were visible, and underground geology was best represented. The deformation region thought to include the Andırap landslide (Yüzer 1971; Ulusay et al. 2008), which is a fossil landslide, is also best observed in these sections. *Fig 4* shows the soil layers and underground geology in Section 1-1 and Section 2-2. In the slopewash on the slope surface, the slope varies from 10^0 to 50^0. In places where slopes are excessive, slopes have been reduced especially since 2014 by palliation and excavation to reduce the sliding forces created by surface waters. Ground movements were measured by placing four inclinometers in Section 1-1. In this section, although mainly impermeable weathering and crushing zones (similar landslide sliding circle (Jung and Verdianz 2013)) are observed at the base, there is a high class rock mass with material parameters (modulus of elasticity, internal friction angle) in general. The other rock unit is the main mass of limestone, and it is the main mass on the left and right banks, dominantly observed close to the surface. Apart from limestone, it is a layer of unit slope wash that forms a geological layer on the surface and covers quite large areas on the left bank of the body, and its thickness is 20-30 meters in places. The slopewash layer generally has low engineering strength properties. Therefore, although both the body and fossil landslides do not directly affect Andırap, surface deformations do occur, especially at the points where the body meets the reservoir water on the left bank. Although the clay soil was generally observed in limestone karstic gaps during dam body excavations, it was observed under a layer of slope wash in Section 1-1 during the HSK-4 inclinometer drilling. Gypsum and anhydride intermediate layers are visible in the clay layer, located on the front near the reservoir, leaning behind a large limestone mass. Section 2-2 was captured over the part of the Andırap region close to the dam and can be considered the boundary of the Andırap landslide on the body side. Here, shale bedrock is the dominant geological unit, and a limestone block sits on it. On the limestone, slope wash deposition is observed on large surfaces. The thickness of the slope wash varies between 10 m and 30 m.

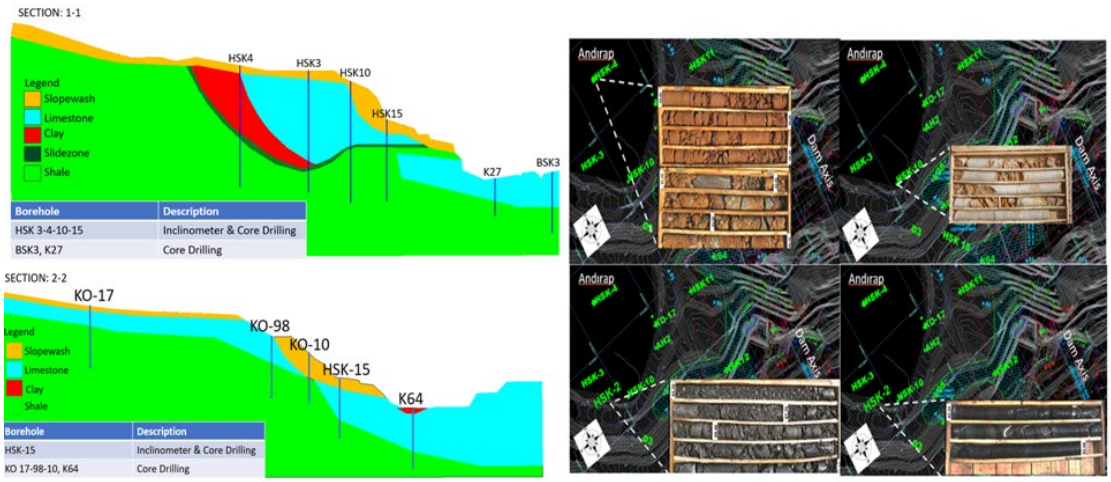

Fig 4. Section 1-1 and Section 2-2 soil stratification and core photos from inclinometers (crossed geological unit samples from HSK-4 (clay:41.10-44.00 m), HSK-10 (limestone 12.5-19.5 m), HSK-2 (transition zone shale 65.2 to 71.35 m), HSK-2 (intact shale rock with high RQD, 82m - 85m)

The clay soil seen in *Fig 4* is 5-10 m thick in places. It has been noted that clay soil occurs by subsea deposition and compression during the period of formation of limestones by tectonic movements during the geological time period observed following the formation of the slope wash layer (Dolsar, 2009). *Fig 4*, a drilling core is shown containing a layer of clay crossed in HSK-4 drilling. Since its engineering strength properties are very low compared to the bedrock mass, it can be defined as the unit that is most easily subjected to deformation as a result of the decrease in shear strength with the increase in pore water pressure formed in it. Despite this, it is believed that it settled down its own without much deformation in the sliding direction since it was located behind a solid limestone mass in the sections that were analyzed.

HSK-10 is a limestone mass obtained from ~~a~~ drilling well, showing slickenside marks and karstic cavity zones due to landslide and tectonism movements (*Fig 4*). A movement was observed especially at the 72m depth in the HSK-10 inclinometer hole during reservoir water filling, and in mid-2015, there was a horizontal displacement of about 72 m, exceed inclinometer pipe capacity. This is confirmed as the boundary of the Andırap fossil landslide as evinced in studies such as Yüzer, 1971 and Doyran and Ulusay 2008. This limestone unit is classified as “Medium Rock" by (Palmstörm et al. 2001; Serafim et al. 1983; Bieniawski, Z.T. 1989).

Karstic cavity zones, stuck clay masses, quartzite and sandstone transitions, dolomitic levels and karstic gaps were observed in the limestone bedrock formation. In the tests, high modulus of elasticity (up to 30 GPa), high shear strength angle (ϕ=60°) and cohesion values of c=1.12 MPa were determined. According to Serafim & Pereira (Palmstörm et al. 2001; Serafim et al. 1983; Bienawski, Z.T. 1989), *Fig 2* shows 30 GPa modulus of elasticity, and according to Serafim and Pereira 1983, Bienawski 1989 (Palmstörm et al. 2001; Serafim et al. 1983; Bienawski, Z.T. 1989) corresponds to a value of about 70 RMR according to the classification and was classified as “good rock” by (Palmstörm et al. 2001; Serafim et al. 1983; Bienawski, Z.T. 1989). This situation is especially manifested by the massive limestones that emerged during the excavation of the main body, as shown in *Fig 2*. The outcropped of the limestone masses, which are massive and do not show any traces of sliding, was seen during the excavation of the upstream cofferdam.

Although the shale basement rock is considered as the mass of the solid rock under the Andırap landslide, it was crushed, crumbled and formed a transition zone that has lower shear strength due to its contact with the limestone bedrock and the movements of the Andırap fossil landslide. Thus, it contains crushed and fragmented zones due to the additional shear stresses caused by its mobility. So the transition zone has a lower internal friction angle and cohesion values relative to the main shale mass, and therefore lower rock strength. Also, the permeability property is also higher than the main shale rock mass. In *Fig 4* shows a photo of the core of the crushed shale bedrock. The shale layer of the crush zone was described as a transition zone and was compressed and crushed between the limestone and the main shale mass at the bottom during the mass movement of the Andırap landslide. It can also be expressed as a zone in which the shear strength is low and the Andırap mass slides over it. The RQD value is below 10 and has a very low modulus of elasticity and RMR values (Palmstörm et al. 2001; Serafim et al. 1983; Bienawski, Z.T. 1989). There is a mass of shale bedrock at depths between 50m and 75m from the surface in the Andırap region. The internal friction angle of this formation is close to ϕ=50^0 and is determined as E=3.1 GPa. This mass of rock, which can be considered a bedrock, has low permeability. The most negative property is that when it meets water, it disperses quickly due to its fine mineral structure (*Fig 4*).

The strength parameters, poisson ratios, modulus of elasticity and unit volume weights used in numerical analyses were determined by using laboratory tests on core and ground samples taken in drilling studies within the scope of the Andırap landslide and pressuremeter tests carried out at the site. Material parameters used in numerical analysis for all units are also presented in Table *2*.

Table 2. Strength parameters used in numerical analysis for geological units

Geological units	Modulus of Elasticity E (MPa)	Poisson Ratio ν (-)	Unit Volume Weight γ (kN/m^3)	Cohesion c (kN/m^2)	Internal Friction Angle ϕ (°)
Slope Wash	40	0.4	205	12	40
Limestone	30000	0.3	27.5	1120	57
Clay	30	0.4	22.5	15	35
Transition zone	1060	0.3	25	240	38
Shale	3100	0.3	26	440	46

NUMERICAL MODEL VERIFICATION

The two sections shown in *Fig 5* considered in the analyses are approximately 70 to 100 m from the Kavşakbendi dam. As part of the Andırap landslide, the ground portion of Section 1-1 was modeled in the Diana FX10.1 finite element program and the displacements occurring between 2014 and 2019 were calculated. The numerical model was validated by comparing inclinometer and surface topographic measurement results and calculation results. In Section 1-1, displacements that are also shown in *Fig 5* respectively took place for the loading condition LC-1 and LC-2. The term TDtXYZ mentioned in these figures refers to the displacement that occurs in case of loading.

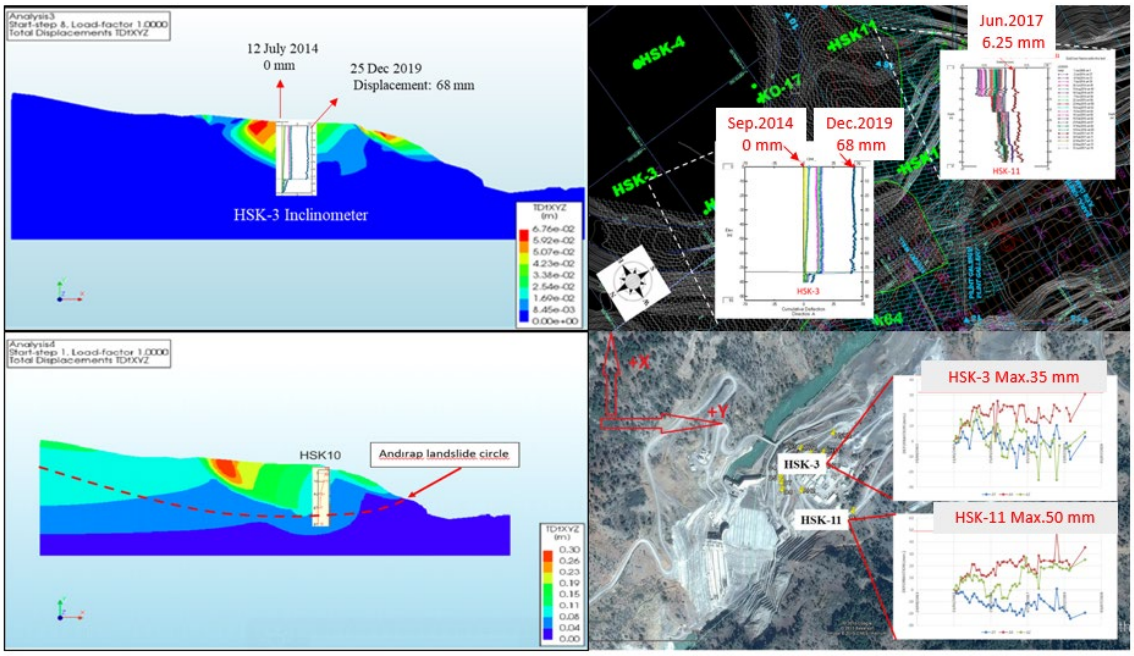

Fig 5. Section 1-1 displacement under LC-1 loading (Diana FX10.1) & measured values in HSK-3 inclinometer, Section 1-1 displacement under LC-2 loading (Diana FX10.1) & measured values (mm) in HSK-10 inclinometer.

In HSK-3 and HSK-10 inclinometer measurements, displacements between 0.04 m and 0.11 m were observed between 67m and 72 m from the surface, especially in the reservoir full state. This depth coincides with the fossil landslide sliding plane mentioned in the limit equilibrium analyses (Jung and Verdianz, 2013) conducted during the design phase of the dam. In later geological periods, this fossil line became stable, undergoing a period of recrystallization and petrification with high cover load pressure. However, as observed in *Fig 4* some drillings carried out along the line, a zone was observed where engineering parameters are low, especially with the impounding phase. According to an analysis by Jung and Verdianz (2013) at the design stage, it was stated that the Andırap landslide would show a displacement of up to 80 mm in the direction of the reservoir after the impounding started and the main reason of this movement would be an increase in the pore water pressure in the Andırap fossil landslide zone containing crushed-sliding zones. As a result, the sliding forces increased and the resistance strength against to sliding decreased, and the upper mass moved towards the reservoir with a decrease in the strength properties of the transition zone under the influence of pore water pressure. This activity reached its maximum in mid-2015 in HSK-10, and at approximately 67m depth, contraction and compression occurred in the HSK-10 inclinometer pipe and the measurements stopped. Measurements taken from HSK-11 and HSK-3 (*Fig 5*) inclinometer holes were taken into account for monitoring movements (deep landslide movements after 2015) occurring in the periods after the impounding phase. According to inclinometer measurements, when the average annual deformation rates were examined, it was found

that movement slowed down and decreased from mid-2015, and almost stopped for the last five years (between 2015-2020). This is in line with the results of the studies carried out before and after the project phase (Doyuran and Ulusay, 2008; Jung and Verdianz, 2013). The most important factor in reducing the speed of movement is that the limestone mass located in front of the landslide mass and continuing from the left bank to the right bank at depths of 54m-57m from the surface increases the forces that counteract the movement. Measurements taken from HSK-3 and HSK-11 inclinometers for about 5 years are shown in *Fig 5*.

The displacement values obtained from stress-strain analyses made with Diana FX10.1 were also compared with surface geodetic measurements taken from the same points and also presented in *Fig 5*. In stress-strain analyses, resultant displacements reaching 0.0676 m were calculated in the landslide area. In *Fig 5*, the geodetic measurement at the HSK-11 location showed a displacement of max. 0.050 m. As shown in the data, the measurements and the values calculated from the numerical analysis are consistent with each other. Geodetic measurements started in 2010 and have been ongoing for about 10 years.

In Section 1-1, as a result of the stress-strain analysis, it was observed that the deformations were concentrated in the slopewash at the end near the reservoir. The reason for this is that due to the partial permeable structure of the slopewash, an increase was observed in pore water pressures and a decrease took place in cohesion with a rise in water content. Thus, the strength of the slopewash material against sliding reduce more quickly than the bedrock material. A stability analysis carried out using the strength reduction method showed that large displacements occurred at sharp points of slopewash. In the case of loading without water in the reservoir (LC-1), a value of 1,150 is obtained for the factor of safety, while in the case of a full reservoir (LC-2), there is a decrease in the strength of the soil by about 15%, and the factor of safety reduces below 1.0. As expected, the main sliding planes and displacements were formed at the slopewash-limestone contact. Inclinometer measurements are quite consistent with the displacements calculated from stress-strain analyses, taking into account the numerical model generated for Section 1-1. Thus, analyses were made for other loading conditions taking into account the verified numerical model.

NUMERICAL ANALYSIS

The stability of the two slopes on the left bank of the Kavşakbendi dam in the Andırap landslide area was investigated by stress-strain and limit equilibrium analyses taking into account different loading situations. In the analyses, loading conditions were taken into account that includes the empty reservoir state (LC-1), the full reservoir state (LC-2) and both the full reservoir state and the effects of earthquakes (LC-3). Possible displacements and factors of safety have been calculated for these loading situations.

Taking into account the above-mentioned loading conditions in two critical sections, stress-strain analyses were performed with Diana FX10.1 finite element program and the displacement and safety of the slope against sliding were investigated. Then, with the Slide6.0 software, which provides a solution using the limit equilibrium analysis method, the numerical analysis was repeated for each loading condition and the factors of safety against sliding were calculated. This way, the factor of safety against sliding determined from stress-strain and limit equilibrium analysis was compared. For the LC-3 loading case, as part of the seismic hazard analysis prepared for Kavşakbendi and HEPP field (Akkar et al. 2009), acceleration values determined by taking into account design spectra with a repetition period of 2475 years were utilized.

Stress-strain analyses

As a result of the stress-strain analysis for the empty reservoir state in Section 1-1 (LC-1) (*Fig 5*), the resultant displacement movement of the X-Y-Z directions occurred in the slopewash and the partially weak clay section, and a displacement of 0.0676 m and a factor of safety against sliding of 1.15 were calculated. The analysis result of the LC-2 loading condition is also shown in *Fig 5*. In this loading condition, resultant displacements approaching 0.15 m were observed on the side of the slopewash, while the clay soil in the middle had displacements approaching 0.30 m, and the factor of safety (F) decreased below 1 under this condition. These displacements formed in slopewash and clay soil are shallow displacements and no global failure (Andırap landslide) occurred. It is observed that the failure planes are developed in the slopewash, where the cohesion value falls, and the clay layer that features pore water pressure due to water intake. The third and final loading condition for Section 1-1 is where the reservoir is full and seismic loading is applied. In the case of LC-3 loading, 0.217 g horizontal and 0.145 g vertical earthquake acceleration values were noted (Akkar et al. 2009). As can be seen in *Fig 6*, a displacement of 0.55 m in the slopewash on the reservoir side and 1.09 m on the clay soil was calculated.

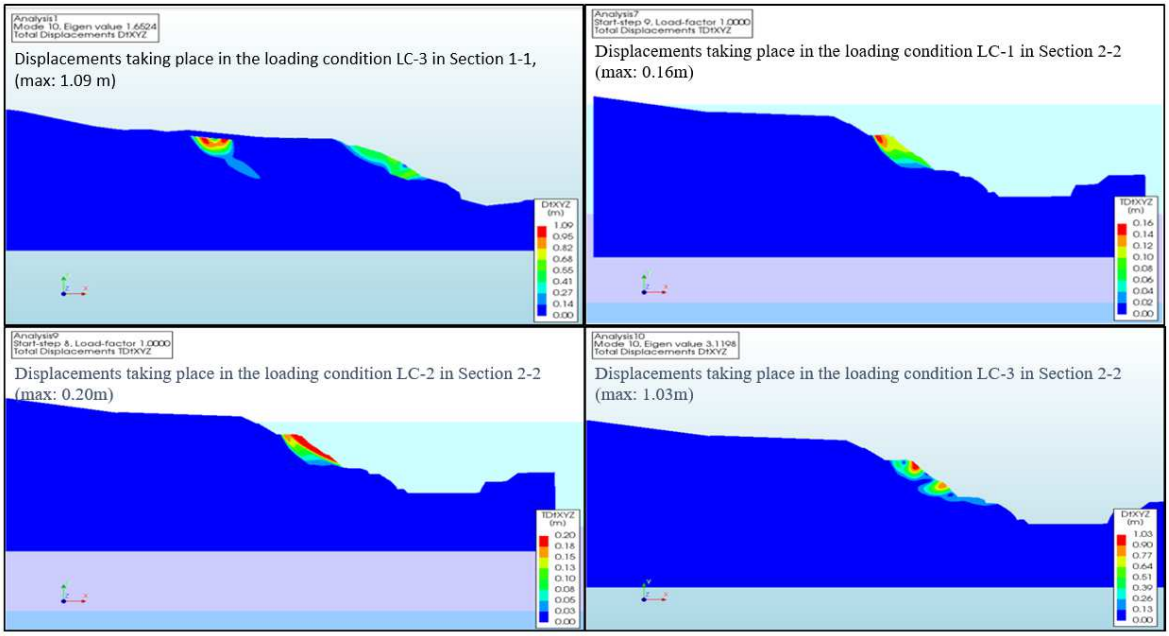

Fig 6.Displacements for different loading conditions in Section 1-1 & Section 2-2

In Section 2-2, if the reservoir is empty (LC-1), the factor of safety was calculated as 1.212 and it was determined that only superficial deformations occur (*Fig 6*). In this section, the thickness of the slopewash ranges from 10 m to 30 m, and surface failures were formed in the slopewash, as in Section 1-1, close to the reservoir. The maximum displacement was calculated as 0.16 m in the slopewash. There is no displacement in the shale and limestone masses classified as other bedrocks. However, the failures are very small-scale in the extreme parts of the slopewash, and the presence of any sliding plane crossing the Andırap sliding plane in a global sense has not been detected.

In Section 2-2, LC-2 loading condition, the displacements increase to 0.20 m by increasing 0.04-0.05 m compared to the empty reservoir loading state (LC-1) (*Fig 6*). In the full reservoir state, shallow surface failures occurred due to the formation of pore water pressure inside the mass of slopewash, as well as the increase in forces moving the slope outward due to the increase in unit volume weight. Although the factor of safety (F) is 1.125 and above 1.0, the factor of safety had decreased with the increase in internal stresses. However, as can be seen in *Fig 6* no global failure is observed.

The displacements determined in the LC-3 loading condition for Section 2-2 are shown in *Fig 6*. In the LC-3 loading condition, 0.217 g horizontal and 0.145 g vertical earthquake acceleration values were considered (Akkar et al. 2009). In this loading, the displacements in the slopewash were calculated as 1.03m. The factor of safety was determined as F<1 for the slopewash. With these displacement increases, the factor of safety reduces below 1.0.

Limit equilibrium analyses

The numerical analyses were repeated in Slide6.0 using the limit equilibrium analysis method and critical factors of safety and sliding circles corresponding to loading conditions were determined and compared with the results obtained from stress-strain analyses. *Fig 7* presents analysis results according to different loading conditions. In *Fig 7*a, a factor of safety of 1.102 was obtained for the LC-1 loading condition and no deep sliding was observed. For the LC-2 loading condition (*Fig 7* b), the factor of safety decreases to 0.993, but again indicates failures in the surface and slopewash. The pore water pressure increased with the entering of water into the slopewash material, and it was determined that there were surface failures in the material due to the weakening of engineering parameters such as the internal friction angle and cohesion of the material. In *Fig 7*c, in the LC-3 loading condition, the sliding circle again crosses through the slopewash, but the factor of safety had decreased to as low as 0.4. As in the stress-strain analysis, surface failures occurred in the slopewash and as a result, the factor of safety decreased to as low as 0.4. There were no signs of failure in the Andırap landslide, which can be considered a fossil landslide. Comparing the results of the analyses, it is observed that the factor of safety 1.102 obtained in the LC-1 loading condition and the 1.15 calculated from the stress-strain analysis are quite close to one another. The locations of the sliding circles obtained from the limit equilibrium analysis and the locations of the main sliding circles in the slopewash obtained from the stress-strain analysis are almost the same. The results obtained from the stress-strain and limit equilibrium analyses show that if there is no water in the reservoir, no global failure will occur.

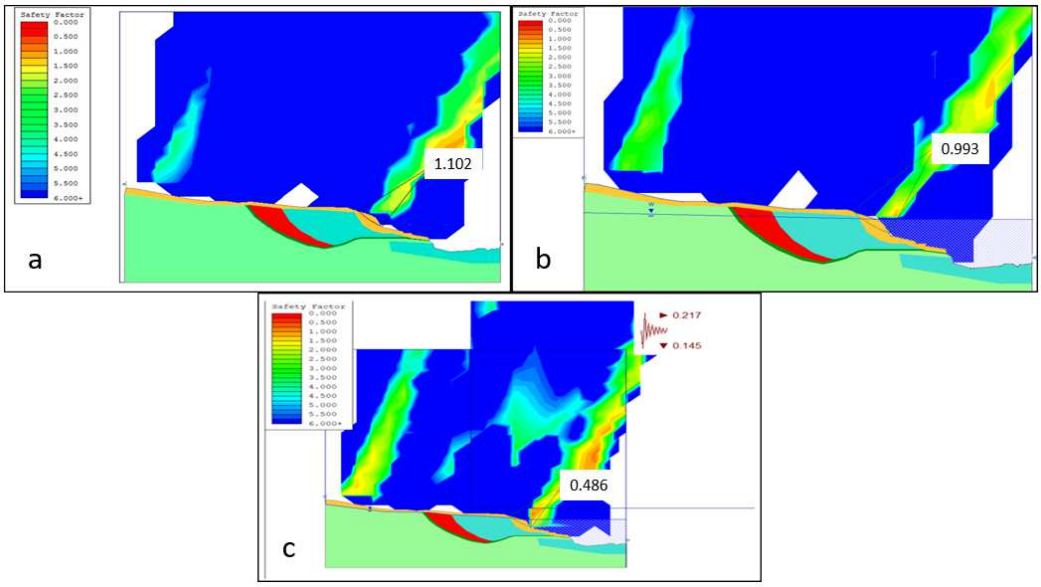

Fig 7. Factors of safety and sliding surfaces obtained for Section 1-1 loading situations a) LC-1 b) LC-2 c) LC-3

Fig 8 shows the factor of safety against sliding and the locations of sliding circles determined for LC-1, LC-2 and LC-3 loading conditions in Section 2-2. In *Fig 8*a for LC-1, the sliding circle crossing through the slopewash and the factor of safety is 1.395, which is close to the value of 1.20 calculated from the stress-strain analysis. As can be seen, no stability problems will be encountered in the LC-1 loading condition. The position of the sliding circle is almost in the same position as the sliding circles obtained from the stress-strain analysis. When the displacements obtained from Diana FX10.1 are examined, it is seen that the displacements approaching 0.10 m, especially in the limestone texture and the slopewash, coincide with the sliding circle in the slopewash found with Slide6.0. In *Fig 8*b, in the LC-2 loading condition, the factor of safety was found to be 1.202 while it was 1.125 in the stress-strain analysis, and the resulting sliding circles cross through the slope wash, as observed in the Slide6.0 analyses. In the LC-3 loading condition, the factor of safety dipped below 1.0 (F=0.759), and similarly, in the stress-strain analysis, the displacement reached 1.03 m. In the analysis, no signs of a sliding circle related to global failure were found (*Fig 8*c). Displacements are concentrated in the promontory parts of the slopewash.

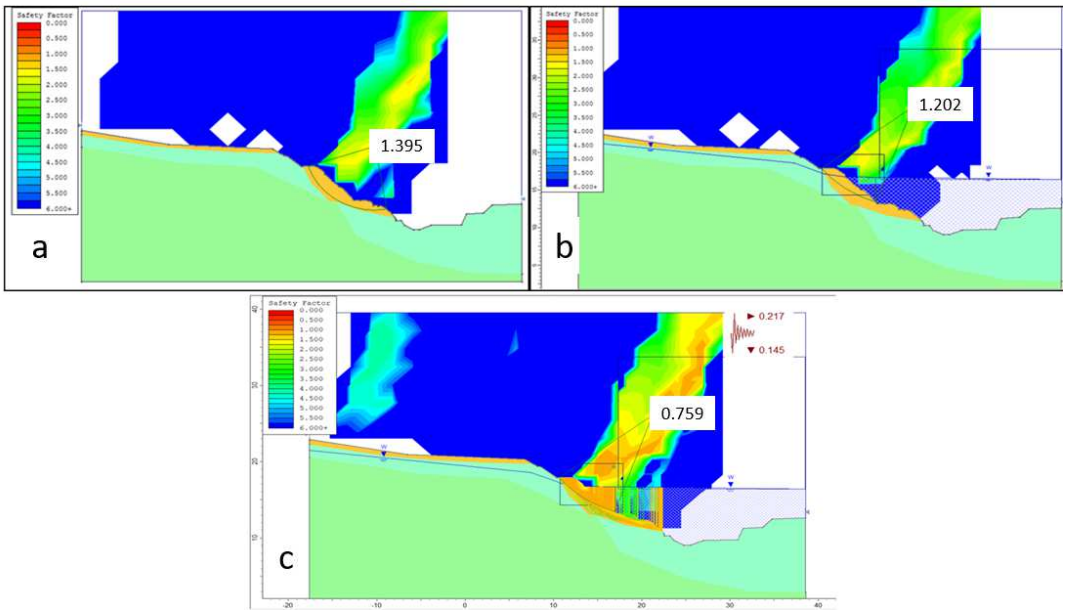

Fig 8. Factors of safety and sliding surfaces obtained for Section 2-2 loading situations a) LC-1 b) LC-2 c) LC-3

Section 3-3 of the Andırap landslide - located 600 m from the dam body - includes 20m-30m thick slopewash. In this section, where the slope wash is quite thick, stability analyses were performed and factors of safety were determined (*Fig 9*). Thus, in the part where the thickness of the slope wash increased, the factors of safety against sliding were examined in the loading conditions of LC-1, LC-2 and LC-3. In this section, the factors of safety against sliding in LC-2 and LC-3 loading conditions were found to be below 1.0. It is noted in the study of Jung and Verdianz, (2013) that this slide, which is considered a deep landslide movement, does not significantly affect the dam body.

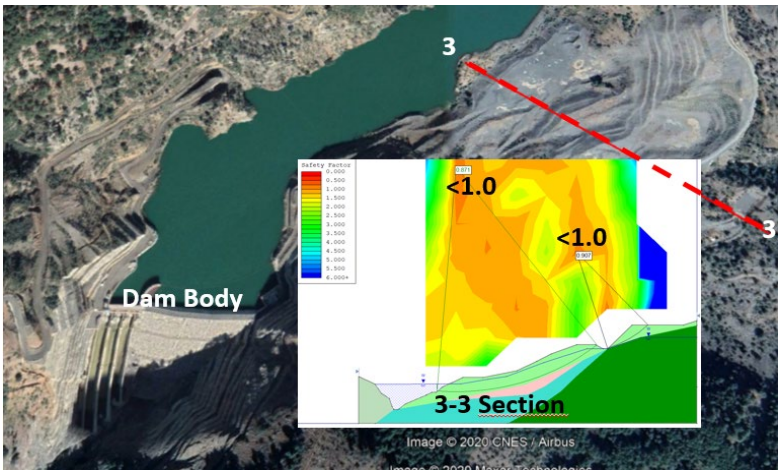

Fig 9. Failures in slope wash under LC2 and LC3 loading conditions in Section 3-3

EVALUATIONS

The results obtained from the numerical analysis conducted at the design stage were presented collectively in *Table 3*. The results of the analysis were summarized by noting shallow failures (in slopewash) and deep failure circles. In deep failures, sliding planes were determined considering the Andırap landslide, for situations where the factor of safety is above 2 in the vicinity of the sliding circle. The displacement value of 0.0676 m calculated from the stress-strain analyses for the Andırap landslide is consistent with inclinometer measurements.

Table 3. Displacement and factors of safety calculated from numerical analyses

Section	Factor of Safety			Displacement (m)		
Section 1-1 (Shallow)	LC-1	LC-2	LC-3	LC-1	LC-2	LC-3
Diana FX10.1	1.15	<1.0	<1.0	0.0676	0.30	1.09
Slide6.0	1.102	0.993	0.486	-	-	-
(Jung and Verdianz, 2013)	1.375	1.23	0.94	-	-	-
Section 2-2 (Shallow)	LC-1	LC-2	LC-3			
Diana FX10.1	1.2125	1.13	<1	0.1600	0.20	1.03
Slide6.0	1.395	1.2	0.759			
(Jung and Verdianz, 2013)	1.996	1.61	0.939			
Section 1-1 (Deep)	LC-1	LC-2	LC-3			
Diana FX10.1	>2.5	>3	>3.5	0.0254	0.08-0.11	0.13
Slide6.0	>3	>4	2.5	-	-	-
(Jung and Verdianz, 2013)	1.83	1.84	1.968	-	-	-
Section 2-2 (Deep)	LC-1	LC-2	LC-3			
Diana FX10.1	>2	>3	3	0.0200	0.05-0.10	-
Slide6.0	>3	5	2			
(Jung and Verdianz, 2013)	3.23	3.86	1.97			
Section 3-3 (Deep)	LC-1	LC-2	LC-3			
Slide6.0	>1.5	>1.3	<1.0			

After stress-strain analyses performed with Diana FX10.1 in Section 1-1 and Section 2-2, limit equilibrium analyses performed with Slide6.0 also found that the sliding circle crossed through the slopewash and the factors of safety were compatible each other. This, in turn, shows that the results are quite consistent with each other for the empty reservoir loading condition. In general, no major displacement had been obtained in the analyses for the Andırap fossil landslide. The reason for this is that although the engineering parameters of the unit defined as a transition zone (sliding zone) is lower than other bedrock properties, it shows no signs of sliding in terms of the factor of safety and displacement in low slope sections (Section 1-1 and Section 2-2).

DISCUSSIONS & RECOMMENDATIONS

In this article, numerical analyses were carried out in two sections representing the site, based on extensive site and laboratory research conducted to study the sliding mechanism of the Andırap landslide mass. After validating the numerical model using site measurements, the behavior of the landslide mass was studied, taking into account the different loading conditions that the dam may encounter during its service life. The results obtained from the numerical analysis are quite consistent with the factors of safety against and the location of the sliding circle calculated in the studies, Jung and Verdianz, 2013 and Doyuran Ulusay, 2008, which were previously conducted in the region. Also, the displacements calculated from deep (inclinometer) and surface (geodetic) monitoring measurements and numerical analyses are quite close to one another. The importance of comparing numerical analysis calculation results with monitoring measurements was also highlighted in Li et al. 2016; Zhou et al. 2016; Bednarczyk et al. 2019; Kaya, A. 2017; and Wang et al. 2020.

The most important factors affecting the analysis results are the stability of the slopes and the resulting displacements, choosing the most appropriate material model and analysis method to model soil behavior by taking into account the loading condition and drainage conditions in the site. The analysis results obtained by taking into account the Mohr-Coulomb failure criterion and the Bishop Simplified and Spencer (Coulomb, 1776) analysis method were consistent with site measurements when the geological units of the landslide mass were evaluated, taking into account the long-term drainage conditions for the zone causing the slide.

While the design of structures such as dams and the stability and deformation analyses of parts affecting the safety of the dam during its service life is being carried out, it is not sufficient to determine the engineering parameters of the materials in light of detailed site and laboratory research. Analysis should also be carried out by taking into account the determined strength parameters, the selected failure criteria, the structure cohesion and analysis method, and the loading conditions that it may encounter during the service period. However, since it is not possible to accurately reflect the section of the underground in the site to numerical analyses, it is necessary to monitor the movements of the soil with deep (inclinometer) and surface (geodetic) measurements and validate the numerical model. After the numerical model is validated, it is possible to be more prepared for possible scenarios with numerical analyses that will be performed taking into account the loading conditions that the slope has not encountered before that day.

CONCLUSIONS

Although the Andırap landslide was first identified in the 1960s, it was active during the dam construction excavations that began in 2009. Due to landslide movements, it was thought that both the project excavations and the process after the dam became operational would stop the progress of the project due to various reasons including the loss of life, and that it could cause the dam body to fail due to the mass slide towards the reservoir during the operation phase. To understand and prevent the mechanism of the landslide, extensive research and measurements were carried out on the site, and in the light of the information obtained, sections representing the site were designated. Material parameters of the formations were determined by using site and laboratory tests. Surface movements were measured, and continue to be measured, with surface topographic measurements and inclinometers made after the beginning of the landslide movement.

As part of the study, stress-strain and stability analyses were performed in two critical sections in the Andırap landslide area located on the left bank of the Kavşakbendi dam. In numerical analyses, the behavior of the slopes was examined by taking into account the empty reservoir state (LC-1), the full reservoir state (LC-2), and the full reservoir state with earthquake effects (LC-3). In the section of slopewash in the upper layer of the landslide, it was determined that the displacement calculated in the cases of LC-1 and LC-2 was 97% consistent with the measurements, while the same figure was about 90% for deep displacements. In the stability analyses, in the case of LC-1 in slopewash, the factor of safety against sliding in Section 1-1 and Section 2-2 was above 1.0, while in the case of LC-2, it was below 1.0 only in Section 1-1. However, the factor of safety in all loading conditions for the deep sliding circle is greater than 1.0. According to stress-strain analyses, global shear was not observed for the sliding circle of the Andırap landslide in any of the loading conditions. In the case of LC-2, the maximum depth displacement was 0.11 m, consistent with the values measured from the inclinometer (0.04 m to 0.11 m). Accordingly, the limestone mass approximately 54m-57m deep from the surface counteracts the movement, and the movement that started from 2010 is thought to have come to a halt over the last decade.

In the Andırap landslide area located on the left bank of the Kavşakbendi dam, it was determined that the monitoring measurements are consistent with the values calculated according to the numerical analyses taken into account in the loading conditions that the dam may encounter during its service life. Thus, it can be said that the numerical model, analysis method and material parameters created for analysis adequately represent the conditions in the natural site conditions. The LC-1 and LC-2 loading conditions correspond to the construction of the dam and the retention of water afterward. The LC-3 loading condition has been a situation yet to be experienced by the landslide mass. Accordingly, under the LC-3 condition, a shallow displacement close to 1m was calculated in the slopewash and the deep sliding displacements were limited to 0.13 m.

According to numerical analyses, no movement is expected that could endanger the operation of the dam and human lives in the area of the Andırap landslide. However, considering that the numerical model may be limited in defining the underground mass of a heterogeneous structure, the movement of the landslide mass should continue to be monitored periodically with deep and surface monitoring devices.

REFERENCES

Abramson L T (2002) Slope Stability and Stabilization Methods, Vol. 2:353-376

Akkar S Yılmaz MT (2009) Site-specific design spectrum of Kavşakbedi weir and hepp site based on probabilistic seismic hazard analysis: 18-26

Bednarczyk Z (2019) Practical Use Of Modern Investigation And Monitoring Methods In Polish Landslide Remediation Projects, Acta Polytechnica CTU Proceedings 23:1–8

Bieniawski Z T (1989) Engineering rock mass classifications. New York Wiley

Coltorti M Tognaccini S (2019) The gravitational landscape of Montespertoli (Valdelsa Basin, Tuscany, Italy): State of activity and characteristics of complex landslides, Geomorphology, 129-142

Coulomb CA (1776) Essai sur une application des regles des maximis et minimis a quelquels problemesde statique relatifs, a la architecture. Mem. Acad. Roy. Div. Sav., vol. 7, pp. 343–387.

Dawson E M Roth W H Drescher A (1999) Slope Stability Analysis by Strength Reduction, Geotechnique, vol. 49, no. 6, pp. 835-840.

Dolsar Engineering I C (2009) Mühendislik Jeolojisi ve Doğal Yapı Malzemelerı Raporu, Kısım B Bölüm 1, 66:19

Doyuran V Ulusay R (2008) Engineering Geological Evaluation Of The Andirap Debris Slide Kavşakbendi Dam Site, Hacettepe University, Middle East Technical University: 5-13

Geoconsult (2009) Geological and Hydrogeological Study: 9-12

Hammah RE Curran J Hyacoub TE Corkum B (2004) Stability Analysis of Rock Slopes using the Finite Element Method. In Proceedings of the ISRM Regional Symposium EUROCK 2004 and the 53rd Geomechanics Colloquy, Salzburg, Austria.

Jung G, Leitner M (2011) Kavsak Bendi Hydroelectrıc Power Plant Guıdelıne Design Material Testing Report 1, 16: 3-15

Jung G, Verdianz M (2012) Material Testing Report 2, 51: 10-50

Jung G, Verdianz M (2013) Andirap Landslide Slope Stability Evaluation Report, 59:10-40

Kaya A (2017) Geotechnical assessment of a slope stability problem in the Citlakkale residential area (Giresun, NE Turkey) , Bull Eng Geol Environ (2017) 76:875–889

Li X Y Zhang, L M F ASCE Jiang, S H Li D Q, ASCE M, Zhou C B (2016) Assessment of Slope Stability in the Monitoring Parameter Space , American Society of Civil Engineers

Özgül N, Kozlu H (2002) Kozan-Feke (Doğu Toroslar) yöresinin stratigrafisi ve yapisal konumu ile ilgili bulgular. TPJD Bülteni, 14 (1): 1-36

Palmstörm A Singh R (2001) The Deformation Modulus Of Rock Masses, Comparisons Between In Situ Tests And Indirect Estimates

Serafim JL, Pereira JP (1983) Consideration of the geomechanics classification of Bieniawski. Proc. Int. Symp. on Engineering Geology and Underground Constructions:1133 - 1144.

Shah C, Sathe S, Mohite B&S (2019) A hill slope failure analysis: A case study of Malingoan village, Maharashtra, India, Geology, Ecology, and Landscapes 7:3-7

Sun G, Huang Y, Li C, Zhenga H (2016) Formation mechanism, deformation characteristics and stability analysis of Wujiang landslide near Centianhe reservoir dam, Engineering Geology 211: 27–38

Wang Z, Xia T, Zhang L, Gao M, Cheng K (2020) Field Geological Investigations and Stability Analysis of Duanjiagou Landslide, International Journal of Geo-Information 18: 5-18

Yüzer E (1971) Seyhan-Kirizli rezervuari jeoloji ve heyelan etüdü. İTÜ, İstanbul 90:20-60

Zbigniew B (2019) Practical Use Of Modern Investigation And Monitoring Methods In Polish Landslide Remediation Projects 8: 3-7

Zhou C, Jiang Q, Wei W, Chen Y, Rong G (2016) Safety monitoring and stability analysis of left bank high slope at Jinping-I hydropower station, Quarterly Journal of Engineering Geology and Hydrogeology, Landslide Research in China 14:3-11