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# Analysis and monitoring of the behavior of a rock fill dam ten years after construction: a case study of the Iran-Madani Dam

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## Abstract

In this study we compared dam monitoring results with those of numerical analysis to propose a plan for the first reservoir impounding of the Iran-Madani Rock fill dam, ten years after the completion of its construction. The stability of the dam body has been assessed using numerical analysis and data obtained from sensors installed in the dam. The correctness and accuracy of the geotechnical parameters of the dam body materials were confirmed by comparing the results of numerical analysis and monitoring through back analysis. The linear correlation coefficients between the experimental data and the numerical results for settlement, pore water pressure, and total stress are 84%, 67%, and 99%, respectively. In addition, the agreement between the design assumptions with both the numerical analysis results and instrumentation data was examined. The arching ratio values obtained from instrumentation and numerical analysis in the core of the dam are 0.47 and 0.35, respectively, indicating the safety of the dam. Finally, a numerical sensitivity analysis was conducted to present a special impounding program for the dam, with a focus on controlling simultaneous changes in pore water pressure and effective stress in the clay core, ten years after the completion of the dam body construction.

**Keywords** Rock fill dam, Monitoring, Pore water pressure, Stress, Settlement, Impounding

## Introduction

Rock fill dams are among the largest embankment structures, and their failure can cause irreparable damage due to the prohibitive costs associated with their construction. Monitoring involves checking the dam's performance at the end of the construction and operation phases and ensuring compliance with design predictions. The International Commission on Large Dams (ICOLD) recommends continuous monitoring of dam safety

and stability during construction and operation (Blind 1983; Cheng 2021). According to embankment dam standards, a back analysis is necessary to adjust material properties and assumptions when there is a significant difference between numerical analysis results and instrumented data (Yu et al. 2007; Wu 2024). Monitoring is achieved by installing appropriate instruments in sensitive areas to measure various parameters such as pore water pressure, deformation, and total stress (Liu 2023). Increased pore water pressure within the dam body reduces effective stress and, consequently, decreases the shear strength of the dam materials, which can lead to dangerous outcomes such as embankment instability (Komasi and Beiranvand 2019; Ghiasi et al. 2021). The amount of saturated settlement in the upstream shell of the dam caused by the first impounding of the reservoir can be determined using data recorded by the dam body sensors (Sorouh et al. 2006; Khalili-Maleki et al. 2022).

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The stability of rock fill dams can be estimated using numerical modeling, which is crucial for controlling the stability of dams against natural disasters (earthquakes, floods, and landslides), poor design, and inadequate maintenance (Farajniya et al. 2022; Sivasuriyan et al. 2021). The stability coefficient obtained from numerical analyses must fall within the allowable range for controlling the stability of dam body slopes (Zhenyu et al. 2020; Ghaemi et al. 2022). Heterogeneous settlement can occur between different parts of the dam, known as "arching" leading to cracks in the impermeable cross-section of the dam body (clay core), especially near the supports, due to the differing properties of core and shell materials. These cracks can expand during reservoir impoundment due to the pressure exerted on the core, potentially leading to hydraulic failure of the dam body (Beiranvand et al. 2019). Rock fill dam cores have been evaluated for hydraulic failure through laboratory tests and numerical analyses, suggesting the use of GM-GC rather than CL materials for rock fill dam cores (Ghanbari et al. 2013; Behrouz Sarand et al. 2023). Studies indicate that maximum long-term settlement typically occurs in the middle of the dam core (Vafaei Poursorkhabi et al. 2023; Mazaheiri et al. 2020). In addition, it has been reported that 88% of total dam settlement occurs during construction (Zareh et al. 2023; Rashidi and Haeri 2017). One study found that settlement decreases and eventually stabilizes over time after the first reservoir impounding, even as the reservoir volume increases (Zhou et al. 2011; Guo et al. 2018). The results of studies conducted thus far indicate that numerical analysis is one of the methods for assessing the stability of rock fill dams. Sole reliance on stability analyses without comparison to results from other methods, such as dam body monitoring, is not a reliable approach. Therefore, this study focuses on the Madani Tabriz dam, which took a decade to construct and has been completed for ten years. To date, no comprehensive study has been conducted on controlling the stability factors of the dam body. Consequently, the stability of the dam body has been assessed concurrently with numerical analysis results and data from precision instrumentation. Furthermore, the correlation coefficients between numerical analysis results and instrumentation data for the three stability factors of the dam body—vertical settlement, pore water pressure variations, and total stress—were examined. Finally, the impounding of the reservoir for this specific dam should not be carried out solely based on conventional codes. Instead, a specialized reservoir impounding plan should be developed according to the dam body conditions, based on sensitivity analysis results, and with simultaneous monitoring of stability factors such as pore pressure variations, stability safety factors, and effective stress values. It is

recommended that for each dam, a specific and tailored reservoir impounding plan be defined and implemented based on the physical and stability conditions of the dam after construction.

### **The geology of the area and the characteristics of the dam**

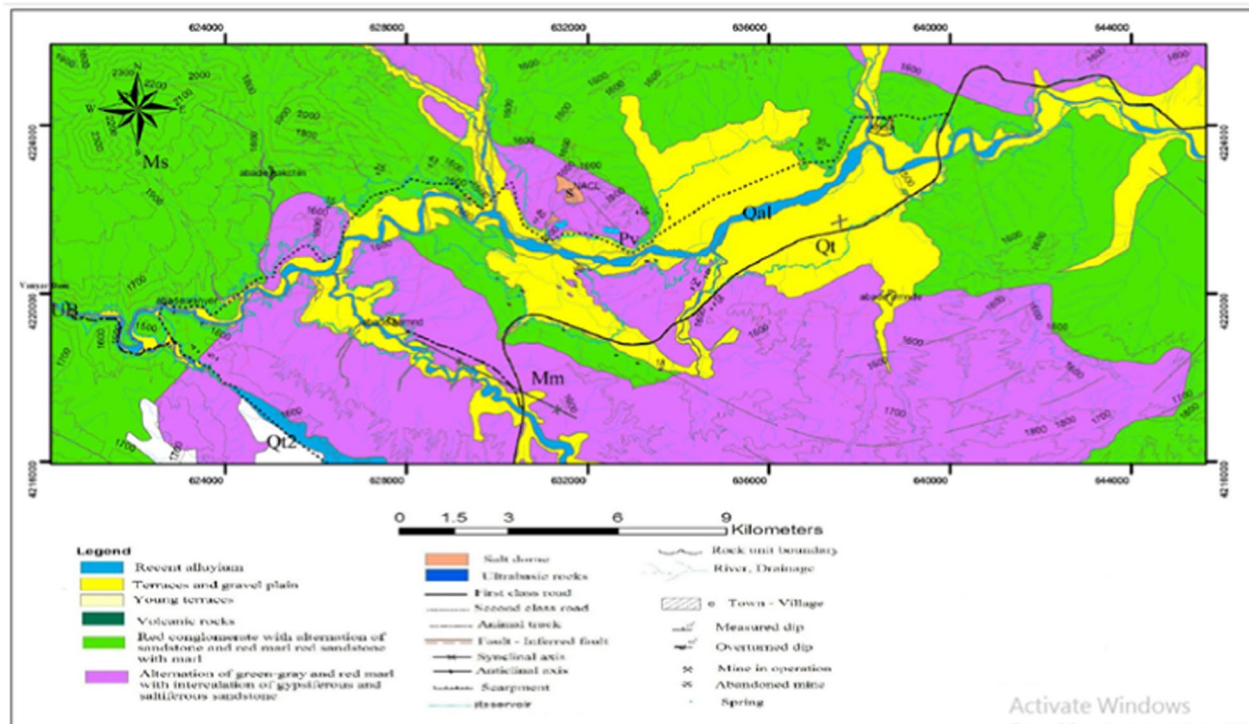
From the point of view of geology, the study area exhibits relatively gentle folding and represents the last phase of Alpine folding from the Pliocene epoch. Significant faults are located near this area, with the most notable being the Tabriz fault, which is situated south of the dam reservoir. The bedrock underlying the foundation areas and the dam axis consists of ultrabasic rocks from the Late Cretaceous period. The quality of the bedrock in this area ranges from poor to very poor. The weathering depth in the bedrock is significant, and there are numerous alteration and fracturing zones. A relatively thick cover of riverbed gravel overlays the bedrock. Figure 1 shows the geological map of the Madani dam reservoir area.

The Madani Rock fill Dam is built on the Aji Chai River in Tabriz, with a crown length of 277 m and a height of 91 m. The crown level of the dam is 1504 m, and the normal water level is 1498 m above sea level. The reservoir volume at the normal level is 361 million cubic meters. Excavation was carried out to a depth of 52 m in the riverbed to reach the bedrock. Two layers of fine- and coarse-grained filters are placed on both sides of the clay core to prevent the leaching of fine-grained materials from the clay core due to water flow through the dam body. The purpose of this dam is to provide freshwater for agricultural use in the Tabriz Plain. The readings from these sensors have been continuously recorded since their installation. Figure 2 shows the plan view and the location of the instruments.

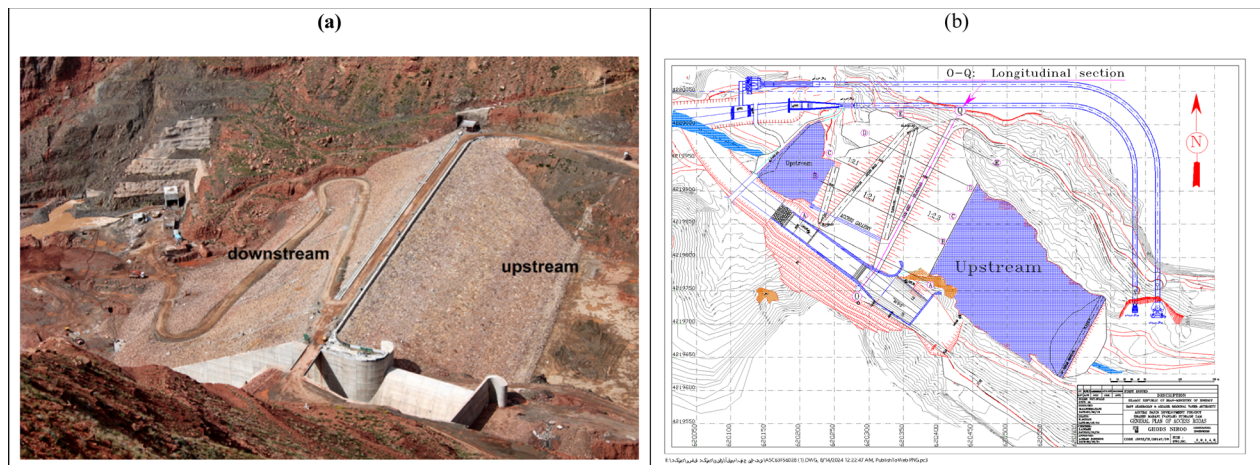
Table 1 presents the technical specifications of Madani Dam and its reservoir (Geological report of Madani dam engineering, 2002).

The primary objective of monitoring the dam body, in addition to ensuring its stability during the post-construction and operational phases, is to analyze and compare the results obtained from instrumentation data with the changes that have occurred relative to the design-stage predictions and existing standards. Another key goal of monitoring is to anticipate potential incidents and prevent any adverse events.

Dam stability was monitored by comparing changes over time to forecasts and existing standards. Based on the dam's characteristics and geological conditions, a monitoring system was implemented to control and manage the performance and behavior of the Madani Dam. Five sections of the dam body were instrumented to monitor the Madani Dam during the design phase.



**Fig. 1** Geological map of Madani Dam Reservoir, Ghods-Niroo consultant engineers Co



**Fig. 2** Dam: **a** Plan view **b** Location of instrumentation

Figure 3 shows the longitudinal section and the largest cross-section.

All the data, including settlement, core pore pressure, and total stress, were collected from over 500 sensors installed in the dam body, ten years after its construction. A total of 120 pressure cells (PC), 62 piezoelectric sensors (VP), 27 piezometers (SP), 17 inclinometers (I), and 275 magnetic settlement detectors (SD) were installed in the dam body. Figure 4 shows the central control of the

electronic equipment and how the manual reading of the mechanical equipment was performed.

**Material properties and numerical modeling**

3D dam bodies and supports were modeled using the Mohr–Coulomb behavioral model and the finite element software MIDAS-GTS.2019V2.1. MIDAS is specialized geotechnical software for analyzing earth and rock fill dams, and its advantages include accurate



**Table 1** Technical specification of the Madani Dam and its reservoir

Dam detail	Value
Height of crest from bedrock (in the largest cross-section)	91m
Height of crest from river bed (in the largest cross-section)	39m
Dam crest elevation	1504m
Normal water level	1498m
Length of dam crest	278m
Width of dam crest	10m
Total dam volume	361Mm <sup>3</sup>
Dam body materials volume	1.7Mm <sup>3</sup>
Dam slope up-stream	1 : 2.3
Dam slope down-stream	1 : 2.1+ Berm
Total reservoir area	12.33Km <sup>2</sup>

modeling, straightforward analyses, and high computational speed.

**Material properties**

The mechanical properties of the materials used in the numerical analyses were obtained from soil mechanics laboratory tests. Table 2 lists the initial material parameters of the dam body based on the Mohr–Coulomb behavior model.

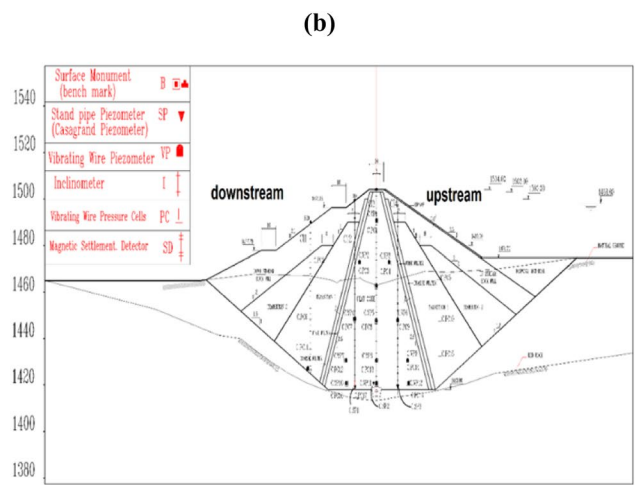
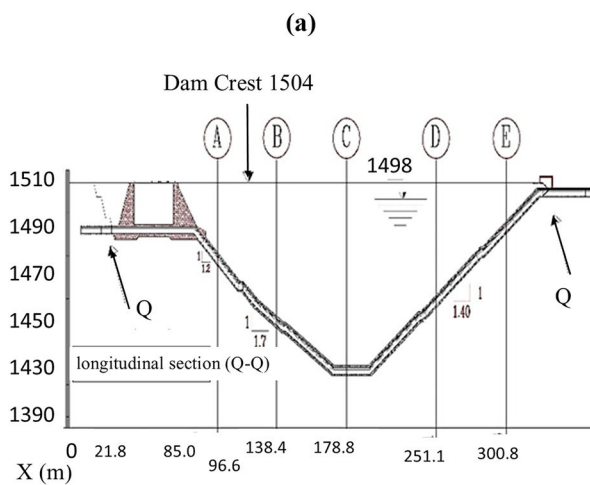
In this table, the parameters,  $\gamma_d \left( \frac{Kn}{m^3} \right)$ ,  $\gamma_{sat} \left( \frac{Kn}{m^3} \right)$ ,  $K * 10^{-6} (m/s)$ ,  $\varphi (deg)$ ,  $C (Kpa)$ ,  $\nu$ , and  $E (Mpa)$  are dry specific gravity, saturation specific gravity, permeability coefficient, internal friction angle, material adhesion, Poisson ratio, and modulus of materials elasticity, respectively.

**Modeling and analyses**

In general, the analysis process in MIDAS software for evaluating the behavior of the Madani Tabriz dam was carried out in three main stages, as outlined below. The results of the analysis were comprehensively used to assess deformation, total stress, and pore pressure:

- Geometric Simulation: Conducted with consideration of boundary conditions to develop the geometric model of the dam.
- Assignment of Mechanical Properties: Involves assigning the mechanical properties of the dam materials and behavioral conditions to provide and allocate soil mechanics characteristics and analytical failure criteria to the geometric model of the dam.
- Mechanical Simulation: Performed to conduct numerical analysis and extract results.

First, the three-dimensional geometry of the dam body, along with its foundation and abutments, is drawn. This 3D geometry is based on the dam’s construction drawings. The main components of the dam, including the core, cutoff wall, filters, and drainage surrounding the clay core, the rock shell, and the transition zones, are depicted in the 3D geometry. Then, the material properties are assigned to each specific region of the dam body, foundation, and abutments for simulation in the software. In this geometric model, the boundary conditions, as well as the horizontal and vertical dimensions of the 3D geometry, are determined and selected based on sensitivity analyses. The Mohr–Coulomb behavioral model is used for numerical modeling. Triangular meshes were employed in this geometric model for numerical analysis, and the mesh size was determined to be unit size



**Fig. 3** Madani Dam: **a** Longitudinal section **b** largest Cross section

**Table 2** List of the initial values material parameters dam body

Parameter	Clay core	Filter	Shell	Transient Shell	Disposal	Alluvial foundation	Bed Rock
$\gamma_d$	19	20	21	20	19.2	18.5	21
$\gamma_{sat}$	20	21	22	21	19.8	19.5	22
$K * 10^{-6}$	0.05	10	100	100	10	100	0.001
$\varphi$	28	33	44	38	28	20	28
$C$	40	0	0	0	20	20	80
$\nu$	0.3	0.25	0.25	0.25	0.3	0.3	0.25
$E$	20	40	70	50	15	40	390

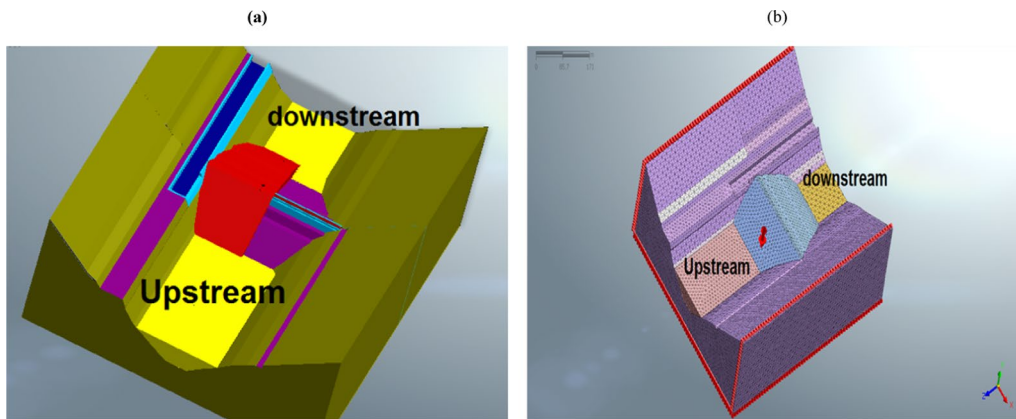
based on the results of the sensitivity analysis. The time duration applied for dam modeling fully aligns with the construction period of the dam, as reported during its construction. In this modeling, the initial displacement of the abutment and the rock foundation was considered zero in the initial stress analysis. For numerical modeling in reservoir impounding analysis and determination of pore pressure in the clay core, the material parameters were defined as homogeneous and isotropic. In other words, the permeability of the materials is assumed to be the same in both horizontal and vertical directions. Numerical modeling was carried out in the following three stages. In the first stage, the riverbed was modeled to determine the residual stress of the bed. Horizontal stress values were calculated using the soil's lateral pressure coefficient. In the second stage, the excavation model was simulated to reach the dam's bedrock from the riverbed in 10 layers. The third stage involved numerical modeling of the dam body in 10 layers, each with a thickness of 6.4 m, starting at the bedrock level and ending at the riverbed level. The construction operation took six years. Considering the location of the river diversion tunnels, the underground water level was assumed to always

be at the riverbed level. The materials of the dam body were saturated below the riverbed, given the duration of construction operations up to the riverbed and the high underground water level in the area. A five-layer model was used to simulate the riverbed up to the dam's crest (1504 m). The construction operation lasted about three years, and an undrained clay core (CU) was assumed. The initial displacement of the supports was considered zero in the initial stress analyses. A mesh size of one (unit) was selected based on the results of sensitivity analyses. Horizontal and vertical boundary dimensions were chosen to create the geometry. Figure 5 shows the 3D geometry and 3D mesh model.

### Back analyses

Back analyses are commonly used in geotechnical engineering to investigate and adjust soil parameters in order to estimate operable material properties in situ. These analyses are popular due to the significant limitations of relying solely on laboratory and in-situ test results for analyzing soil profiles. Dam body stability can only be accurately controlled using realistic material parameters

**Fig. 4** Madani Dam: **a** Electronic equipment control room **b** Manual reading



**Fig. 5** Madani Dam: **a** 3D geometry model **b** 3D mesh of the computational domain

and precise modeling. This research aims to verify the accuracy of the parameter values used in the numerical analyses of the dam body materials by examining the correlation coefficients between the instrumentation data and the numerical analysis results.

**Results and discussion**

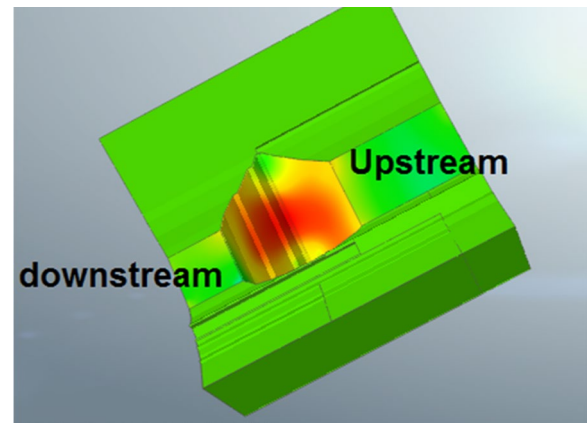
The results of the numerical analyses and the stability safety factor of the dam body at various reservoir impoundment levels are presented below.

**Settlement**

The settlement of embankment and rock fill dams is a significant factor in dam stability. The weight and density of materials during construction impose additional load on the underlying layers, leading to internal deformation. A series of magnetic plates were placed at different depths inside the settlement tube to measure the deformation of the dam body. The base plate was installed at the lowest level on the rock bed, where the least movement occurs, making this point nearly fixed. A total of 17 inclinometers (I) and 275 magnetic settlement detectors (SD) were used at different levels of the dam body to assess the deformations of the Madani Dam. Figure 6 shows the vertical settlement of the dam body resulting from the numerical analyses.

In addition, Table 3 presents the vertical settlement values obtained from the magnetic settlement detectors and numerical analysis across five cross-sections of the dam.

The maximum vertical settlement of the Madani Tabriz dam body ten years after the completion of its construction is 1.90 m based on instrumentation data and 1.79 m based on numerical analysis. The vertical settlement recorded by instrumentation across all cross-sections of the dam body is greater than that predicted by numerical analyses. This discrepancy may be due to the adequate



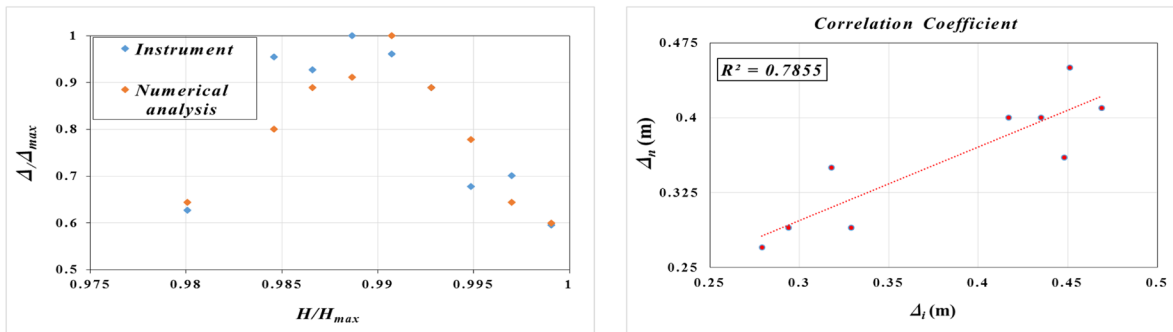
**Fig. 6** Contour of vertical settlement obtained from numerical analysis

**Table 3** Maximum vertical settlement in cross-sections A, B, C, D, and E

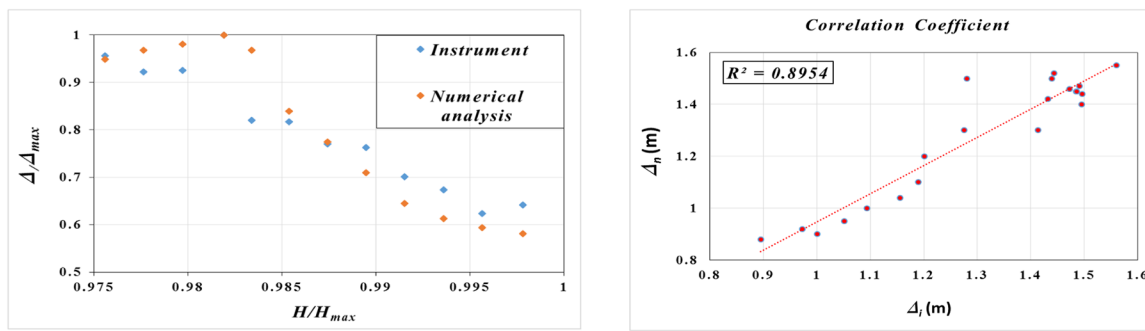
Cross-section	E	D	C	B	A
Maximum settlement numerical analysis (m)	1.1	1.3	1.79	1.55	0.45
Maximum settlement instruments (m)	1.17	1.33	1.90	1.56	0.469

consolidation of the dam’s clay core materials over the past ten years and the occurrence of several natural earthquakes in the vicinity of the dam, which could have contributed to the compaction of the dam’s body materials during this period. Figure 7 shows the curve of settlement relative changes ( $\Delta/\Delta_{max}$ ) in cross-sections A, B, C, D, and E to determine the dimensionless multivariate

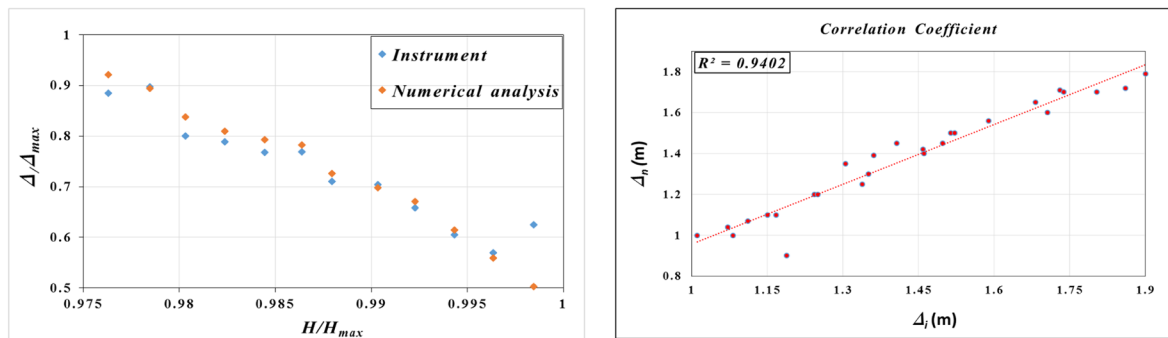
(a): Sec (A)



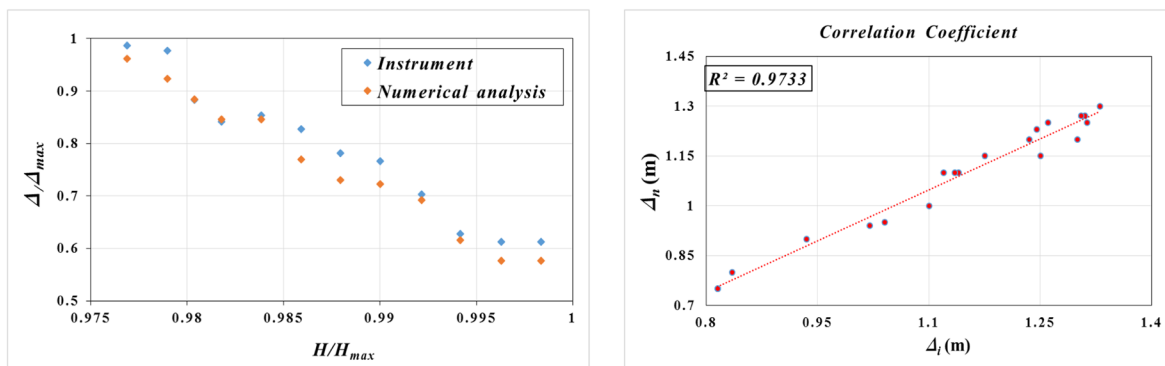
(b): Sec (B)



(c): Sec (C)



(d): Sec (D)



**Fig. 7** Settlements curves comparison and Correlation Coefficients in cross-sections A, B, C, D, and E

(e): Sec (E)

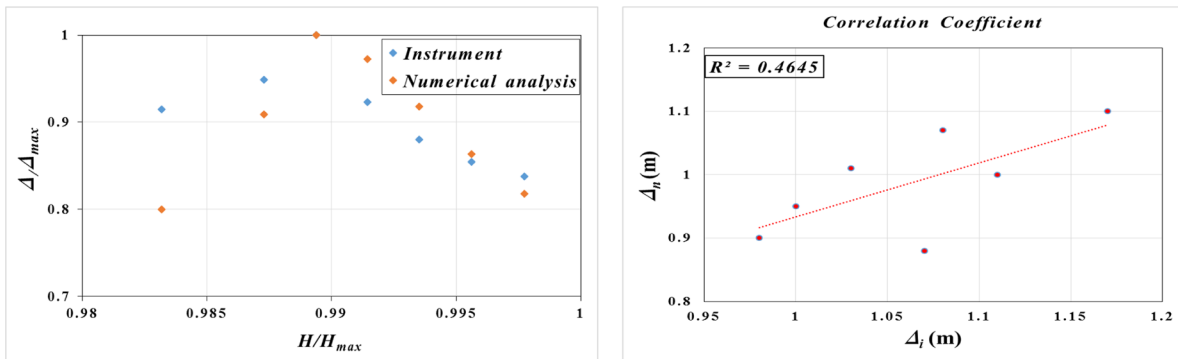


Fig. 7 continued

relationship between settlement, total stress, and pore water pressure. The correlation coefficient of the settlement data obtained from the precision instruments and numerical analyses was greater than 83.8%.

In Fig. 7,  $H$  is the level at which settlement was measured (level of settlement plates),  $H_{max}$  is the maximum dam crown level,  $\Delta$  is the Settlement, and  $\Delta_{max}$  is the maximum settlement.

### Vertical stress

Pressure cells record the variations in total stress in different parts of the dam body and measure the stresses in three directions (two horizontal and one vertical). In the Madani Dam, 120 pressure cells were installed in various parts of the structure. Table 4 shows the total vertical stress values obtained from the pressure cells and numerical analyses.

In Table 4, PC is the Vibrating Wire Pressure Cell which were installed in 5 different cross sections of the dam.

The maximum vertical stress occurs at the lower levels of the dam body and the central regions of the clay core. Due to the trapezoidal cross-sectional geometry of the rock fill dam body, the vertical stress is concentrated at the mid-height of the clay core. Based on instrument readings from pressure cells and numerical analysis, the maximum total stress at the interface between the core and the foundation is 1205 kPa and 1440 kPa. In all instrumented cross-sections of the dam body, the total stress measured by the pressure cells is less than that obtained from numerical analysis. This discrepancy may be due to the lower compaction of the materials surrounding the pressure cells during the dam's construction compared to other parts of the clay core. The reduced compaction around the pressure cells was likely intentional to prevent damage to the cells during construction activities. Figure 8 shows the curve of stress changes ( $\sigma/\sigma_{max}$ ) and correlation coefficient of the total stress values

obtained from the precision instruments and numerical analyses. The correlation coefficient of the total stress data obtained from precision instruments and numerical analyses was more than 99.09%.

In Fig. 8,  $H$  is the Level of the plates,  $H_{max}$  is the maximum dam crown level,  $\sigma$  is stress, and  $\sigma_{max}$  is the maximum Stress.

### Pore water pressure

Pore water pressure is a critical issue in geotechnical engineering. To determine pore pressure in the Madani Dam, both electric and Casagrande piezometers were used. Excess pore pressure in the dam's body has been generated due to the wide clay core, the presence of overburden, low permeability of the core, and proper material compaction during construction. The maximum pore water pressure was recorded in the lower and middle levels of the clay core ten years after construction (at the core-to-foundation interface). Table 5 presents the pore water pressure in cross-sections C, B, and D obtained from electric piezometers (VP), Casagrande piezometers (SP), and numerical analyses.

The maximum pore water pressure recorded by the piezometers and obtained from numerical analysis is 420 kPa and 450 kPa, respectively. Due to the substantial width of the clay core and the saturation of the lower levels of the core from the high groundwater level and river water level, the maximum pore water pressure ten years after the completion of construction has occurred in the lower levels and central part of the clay core (at the core-foundation interface). Over time, with the proper consolidation of the clay core materials, a very good correlation between the pore pressure data has been observed. The  $A_r$  value, the arching ratio obtained from instrument monitoring data and numerical analysis, is 0.35 and 0.47, respectively, which is close to the design stage value of 0.5. Based on these



**Table 4** Vertical stresses in cross-sections A, B, C, D, and E

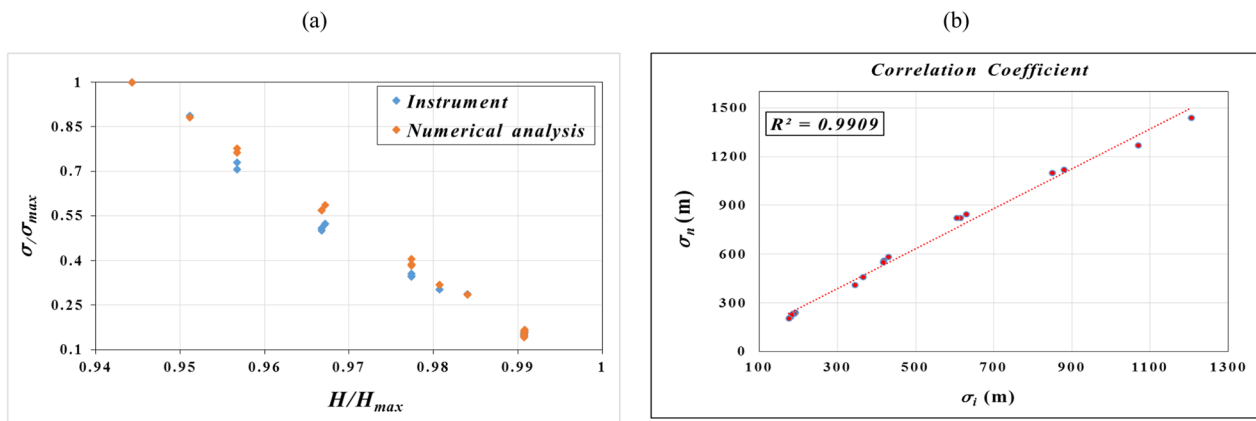
Cell No	Location	Distance to Axis (m)	Install Elevation (z)	Vertical total stress	
				Instruments	N. analysis
APC.1	Core center	0.74	1490.070	181	213
APC.2	Down-stream core	-6.685	1475.366	365	460
APC.3	Up-stream core	6.754	1475.9	351	445
BPC.1	Core center	0.075	1490.1	189	234
BPC.3	Down-stream core	-9.546	1469.794	419	559
BPC.4	Up-stream core	10.302	1469.794	413	552
BPC.7	Down-stream core	-17.061	1454.338	565.0	790
BPC.8	Core center	0.36	1454.457	615.0	820
BPC.9	Up-stream core	18.025	1454.489	580	800
BPC.11	Down-stream core	-16.2	1439.205	805	1095
BPC.12	Core center	0.08	1439.164	880	1120
BPC.13	Up-stream core	16.81	1439.161	845	1100
CPC.1	Core center	0.5	1490.11	192.4	240
CPC.3	Down-stream core	-10.541	1470.036	430	583
CPC.4	Up-stream core	10.245	1469.935	419.3	579
CPC.7	Down-stream core	-16.926	1454.649	575.0	805
CPC.8	Core center	0.492	1454.66	630.18	845
CPC.9	Up-stream core	17.957	1454.629	591.62	812
CPC.12	Down-stream core	-20.023	1430.909	1008.3	1250
CPC.13	Core center	0	1431	-	1270
CPC.14	Up-stream core	20.312	1431.321	1070.5	1255
CPC.16	Down-stream core	-20.43	1420.294	1205	1425
CPC.17	Core center	0.121	1420.55	1150	1440
CPC.18	Up-stream core	19.841	1420.296	1160	1420
DPC.1	Core center	0.4	1490.11	184	226
DPC.3	Down-stream core	-10.692	1470.11	417	551
DPC.4	Up-stream core	10.779	1470.159	408	544
DPC.9	Down-stream core	-17.917	1454.422	560.0	785
DPC.10	Core center	0.352	1454.488	605.0	820
DPC.11	Up-stream core	17.965	1454.17	575	790
DPC.6	Down-stream core	-17.15	1439.158	790	1080
DPC.7	Core center	0.18	1439.044	850	1100
DPC.8	Up-stream core	17.07	1439.129	822	1070
EPC.1	Core center	0.049	1489.96	175	204
EPC.2	Down-stream core	-6.88	1479.846	345	411
EPC.3	Up-stream core	6.73	1479.907	321	404

results, the likelihood of arching and hydraulic fracturing of the dam body due to the wide core is not feasible. Figure 9 shows the curve of pressure changes ( $P/P_{\max}$ ) and the correlation coefficient of the total pressure data obtained from the piezometer instruments and numerical analyses. The correlation coefficient of the pore pressure data obtained from the piezometer instruments and numerical analyses is more than 67%.

In Fig. 9,  $H$  is the plate level,  $H_{\max}$  is the Maximum dam crown level,  $\rho$  is the Pore water pressure, and  $\rho_{\max}$  is the maximum pore water pressure.

In this section we used 27 data with a correlation coefficient above 89%. Multivariate regression presented a relationship between the relative changes of the settlement (dependent variable) based on the relative changes of the pore water pressure and the total stress.

Equation (1) shows the relation of relative changes of the settlement.



**Fig. 8** a Vertical total stress changes b Correlation coefficient between precision instruments and numerical analysis values of total stress

**Table 5** Values of pore water pressure in cross-sections B, C, and D

Piezometer No	Location	Distance Axis(m)	Install Elevation (z)	Pore pressure	
				Instruments	N. analysis
BVP.7	Down-stream core	-17.36	1439.18	358.6	375
BVP.8	Core center	0.48	1439.16	360.2	380
BVP.9	Up-stream core	16.49	1439.22	366.5	385
BSP.1	Down-stream core	-15	1438	175	380
BSP.2	Core center	-3.19	1438	161	383
BSP.3	Up-stream core	13.38	1437	195	395
CVP.10	Down-stream core	-20.6	1420.52	340	402
CVP.11	Core center	0.422	1420.58	328	396
CVP.12	Up-stream core	20.25	1420.29	347	414
CSP.1	Down-stream core	-16.24	1416.7	403	438
CSP.2	Core center	-3.80	1416.7	392	425
CSP.3	Up-stream core	15.9	1416.7	420	450
DVP.4	Down-stream core	-16.42	1439.14	377.7	355
DVP.5	Core center	0.12	1439.02	352	363
DVP.6	Up-stream core	17.1	1439.1	361.1	380
DSP.1	Down-stream core	-14.91	1438	96	358
DSP.2	Core center	-2.92	1439	86	363
DSP.3	Up-stream core	14.85	1439	86	380

$$Y = \frac{\Delta}{\Delta_{max}} = f\left(\frac{\sigma}{\sigma_{max}}, \frac{\rho}{\rho_{max}}\right) \quad (1)$$

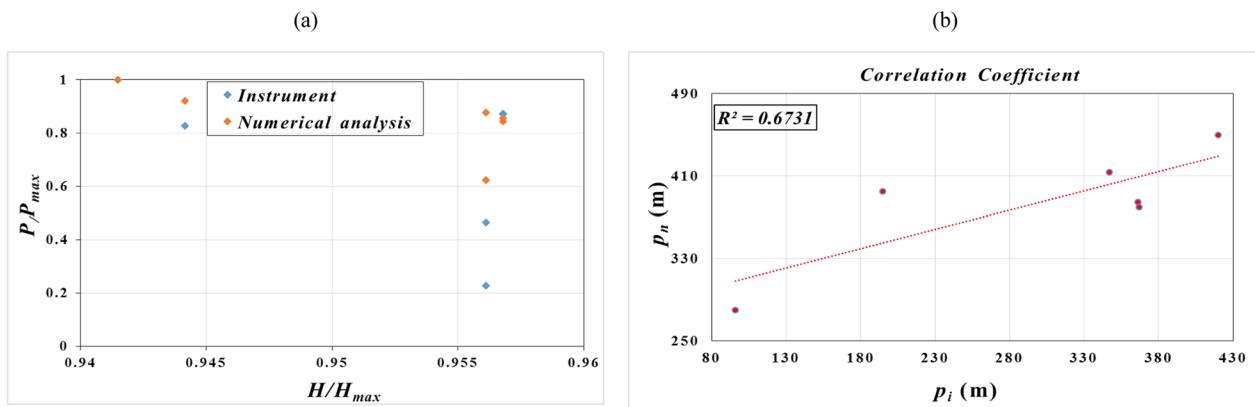
variable), and  $\left(\frac{P}{P_{max}}\right)$  is the relative pore pressure changes to maximum pore water pressure (independent variable).

$$Y = \frac{\Delta}{\Delta_{max}} = 0.807 - 0.0011795\left(\frac{\sigma}{\sigma_{max}}\right) - 0.0582\left(\frac{\rho}{\rho_{max}}\right)$$

where, Y is the relative changes of settlement to maximum settlement (dependent variable),  $\left(\frac{\sigma}{\sigma_{max}}\right)$  is the relative changes of stress to maximum stress (independent

**Back analyses**

The validation of the actual material parameters used in the dam body is determined through back analysis. The correlation coefficients between the instrument data and



**Fig. 9** a Pore pressure changes b Correlation coefficient between numerical and experimental values of pore pressure

numerical analysis results for total stress, pore pressure, and settlement are 99.09, 67.31, and 83.8, respectively. The results indicate a very strong correlation between the numerical and experimental data.

#### Reservoir impounding program

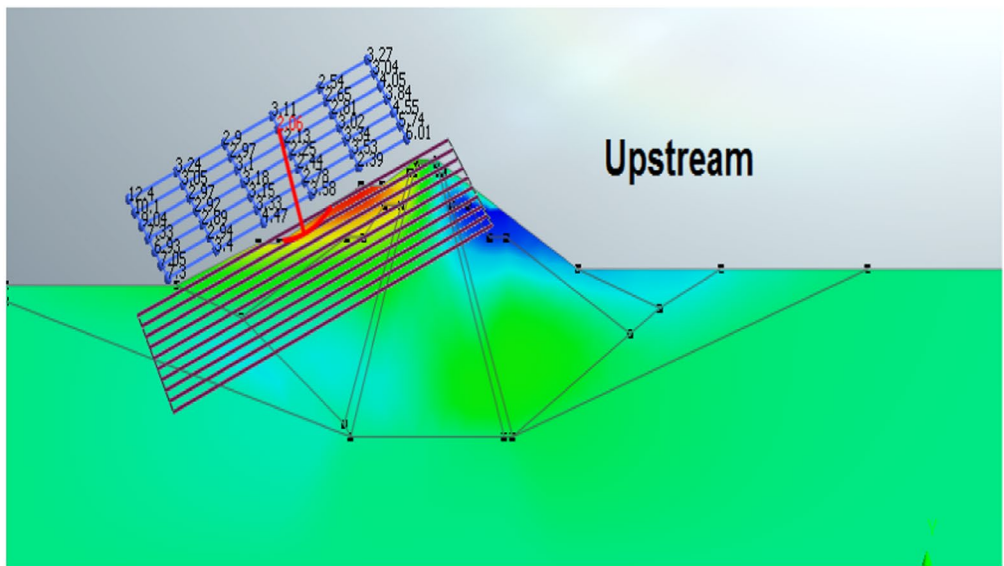
Considering the unique conditions of the Madani Tabriz rock fill dam, where the construction of the dam body took ten years and another ten years have passed since its completion, the reservoir impounding process will not follow conventional procedures. Therefore, a safe and specialized impounding plan for the Madani rock fill dam must be developed based on the dam's specific conditions, including sensitivity analyses that involve numerical stability analyses assuming reservoir impounding at different levels. Simultaneous monitoring of key stability factors, such as impounding time, pore water pressure, stability safety factors, and effective stress, is essential. Furthermore, it is necessary to maintain appropriate monitoring of the dam's stability during the reservoir impounding and operational phases by using data from settlement gauges, inclinometers, pressure cells, and electric piezometers installed at five cross-sections of the dam body. Based on this approach, it is recommended that for any dam that has not yet been impounded or is still under construction, a specialized impounding plan tailored to the specific conditions of that dam including the construction period and other factors affecting dam stability should be developed. Figures 10 and 11 show the dam body stability factors considering the time and different impounding levels, respectively. Figure 12 illustrates the changes in effective stress based on different reservoir impounding levels.

Figure 11 shows that the dam reservoir can be fully impounded in 80 days with a controlled filling rate of 30 cm per day. As the reservoir is filled, the clay cores become saturated, and water is expelled from the clay

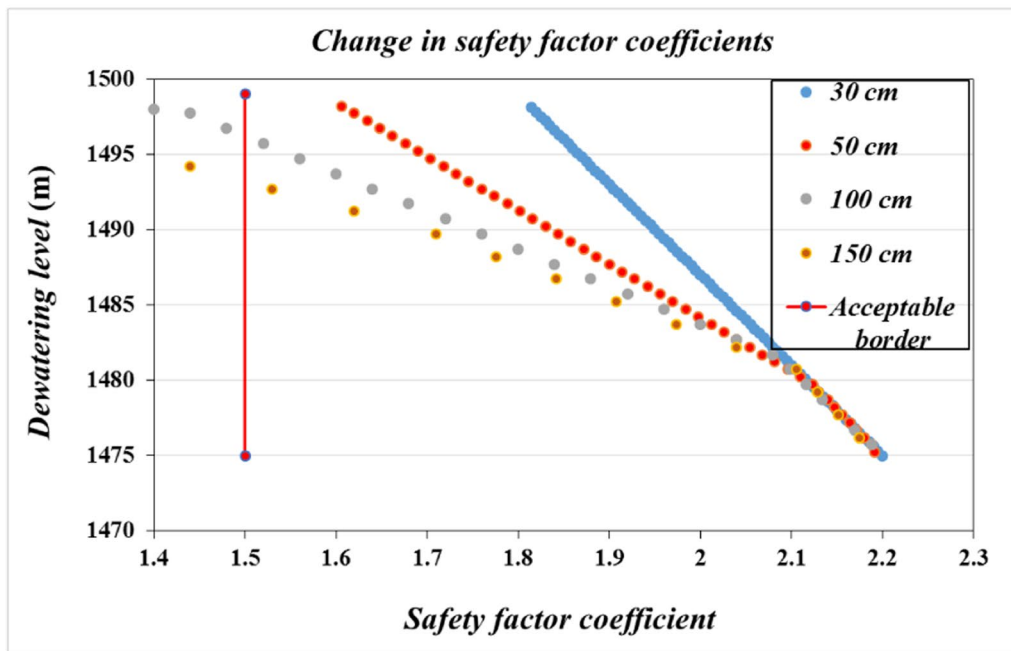
cores during this period. Under these conditions, the pore water pressure in the core and the dam body's stability factor remain within acceptable limits. When the water level in the reservoir rises, the rate of pore pressure increase in the core slows down, and the rate of effective stress reduction decreases. In Fig. 12, the changes in pore water pressure and effective stress in the core at two filling rates, 30 and 150 cm per day, are compared. Sensitivity analyses of the impounding process were conducted, and it is recommended that the reservoir be filled at a rate of 30 cm per day.

#### Conclusion

The maximum vertical settlement, based on instrumentation readings and numerical analyses, is 190 cm and 179 cm, respectively. The maximum vertical stress at the core-foundation interface, obtained from pressure cell readings and numerical analysis, is 1205 kPa and 1440 kPa, respectively. The maximum pore water pressure recorded by installed piezometers and from numerical analysis is 420 kPa and 450 kPa, respectively. The correlation coefficients between settlement data, pore water pressure, and total stress, derived from instrumentation and numerical analyses, are 0.67, 0.84, and 0.99, respectively, indicating a good agreement between the data. It is recommended to control the impounding rate of the reservoir to prevent increased pore water pressure during the impounding process and to avoid hydraulic failure. The purpose of back-analysis is to validate the parameters used for the materials in earth and rock fill dams. Based on the instrumentation data for the Madani dam body and numerical analysis results, it can be concluded that the dam's instrumentation system provides accurate measurement and recording. According to the back-analysis results, the parameters used for the materials, obtained from laboratory tests during dam construction, are accurate and reliable. Sensitivity analysis of the



(a)



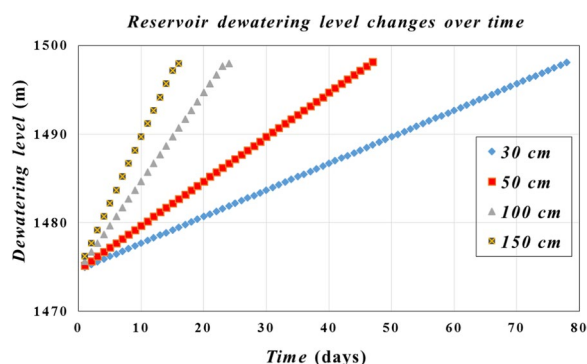
(b)

**Fig. 10** a Changes stability coefficients b levels of impounding the dam reservoir

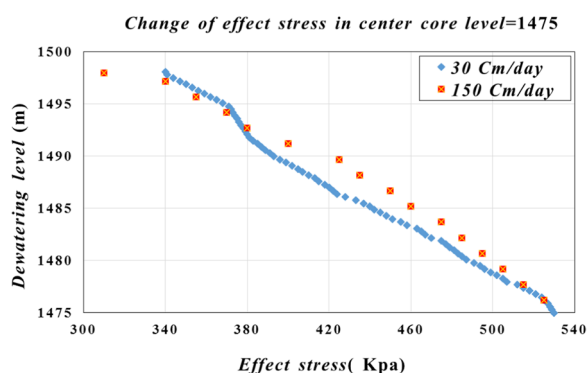
reservoir impounding at different levels until reaching the normal reservoir level was conducted. For this particular dam, given that ten years have passed since the dam body was completed, with a maximum impounding rate of 30

cm per day, pore pressure in the core can be controlled, and by monitoring the stability safety factor and effective stress, the reservoir will be safely filled within 80 days. Therefore, it is recommended that for each earth or rock





**Fig. 11** Changes reservoir level of dam based- period time



**Fig. 12** Variation of effective stress and levels of impounding the reservoir of dam

fill dam, a specific impounding plan should be developed, considering all safety aspects for the dam.

**Author contributions**

R.F. carried out the reliability assessments and calibrated the finite element model. R.V.P. wrote the main manuscript except for the section “experimental measurements”. R.F and A.Z. carried out the experiments and wrote the section “experimental measurements”. R.D. control and edit main paper. All Authors reviewed the manuscript.

**Availability of data and materials**

No datasets were generated or analysed during the current study.

**Declarations**

**Competing interests**

The authors declare no competing interests.

Received: 26 March 2024 Accepted: 15 September 2024

Published online: 08 October 2024

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