Numerical Analysis and Back Calculation of Embankment Dam Using Monitoring Results (Case Study: Iran-Lurestan Rudbar)

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Abstract:

In this paper, comparing the results obtained from monitoring and numerical modeling carried out to significantly help to analyze the stability of embankment dams. One of the aims of this research is a comparison of these analyses with the fact they are the precise instrument results, in addition to validating the numerical analysis, is the basis for performing a back analysis to obtain accurate Geotechnical parameters. Rudbar Lurestan dam was selected as a case study and the results of its instrumentation were processed and analyzed. The model was modeled in a static state at the time of construction. FLAC2D V7 software was used for modeling and simulation was done using Mohr-Coulomb behavioral model and the initial parameters of the material (Obtained from the laboratory). Then, after completing the modeling at the end of construction and initial impounding the results of back calculation, the final parameters of the materials were obtained and conformity of the results of these analyses with the design assumptions and the instrumentation results was evaluated. The observations show that there is a good agreement between the monitoring results of the modeling and also final parameters obtained from the back calculation show a growing trend in most parameters.

Keywords: Back Calculation, Monitoring, Instrumentation, FLAC 2D, Rudbar Rock fill Dam.
1. Introduction:

Earth and rock fill dams are important and huge structures. The initial material construction of these dams is made of natural materials such as earth or rock. The behavior of these dams cannot be predicted in a well-defined and specific framework because of their three-phase nature (soil, water, air), soil properties of materials as well as due to the high cost of dams and heavy damages caused by their insecurity, control of the stability and monitoring of the dams during construction, impounding and exploitation are very important. On the other hand, designing and accurate modeling of the dams is also very important. If you can find a behavioral model with realistic parameters, which is consistent with the instrumental results, this model can be used to study the behavior of the dam in sections where the instrumentation is not installed or damaged. Also, it may problems during impounding or exploitation. Therefore, with proper modeling of the plan, dam safety can be ensured against accidents such as hydraulic failure and arching ratio as well as the actual Geotechnical parameters of the dam body can be obtained during construction, impounding and operation.

Monitoring operations and studying the process of data changes obtained from the instrumentation used in the dams, in many cases, prevent the dam from destruction by predicting the occurrence of lateral and linear cracks, arching, hydraulic failure and increasing stress; and also will reduce possible damages to downstream of the dam. Therefore, control of dam behavior during construction, impounding and exploitation is very necessary. In recent decades, sustainability of the dams has been particular attention to engineers.

One of the recommendations of the International Commission on Large Dams is the constant control of safety and sustainability of dams during construction and exploitation [1]. The dams can be modified based on displacement monitoring data and penetration assessments. Park and Oh [2] discussed about modifying dam method using the low-pressure injection method. To reduce the impact of rock falls, the protection of structures such as reinforced concrete, stone vents, retaining walls, rigid and flexible dam, and dams are widely constructed in mountainous areas [3, 4].

Statistical information show that earth dams are exposed to destruction more than concrete dams, also more than 50% of destructions or damages of earth dams were at the time of construction and during the initial impounding [5]. The observation data of embankment dam displacement can reflect the dam’s actual service behavior intuitively [6]. Monitoring and surveying of construction and structures are done with important goals, as a key sign in the implementation of constructive and Geotechnical projects such as construction control, warning against any imminent failure, quality assurance, validation of new theories, resolving legal and judicial matters as quickly as possible, verifying long time implementation and many other reasons [7], and also use of types of Geotechnical instrumentation that are one of the most commonly used methods in the monitoring of Geotechnical structures, especially in earth dams [8]. Engineering and economic researches in the building of dam construction projects around the world show that, in many cases, rock fill dams with a clay core are impervious and are considered as the best choice for the ultimate design [9]. Most dam failures experienced a process of quantitative to qualitative changes [10].
Mahin Rousta [11] evaluated (appraised) linear behavior of rock fill materials using numerical analysis and laboratory test results. Strain hardening and softening model in the FLAC software were modified based on data provided by laboratory examinations in order to estimate the failure phenomenon result from the settlement in the earth rock fill dams at the time of the flood. The obtained results helped the dam's engineers to have a better predication of the nonlinear behavior of the failure phenomenon from the result of the settlement in the upstream crust.

The pore water pressure, stresses, and settlement are the main parameters for the stability analysis of earth and rock fill dams [12], which have been investigated in this study. Beiranvand and Komasi [13] analyzed the pore water pressure on the Eyvashan earth rockfill dam in the initial impounding by two Plaxis and Geostudio software's. In addition, the data recorded by the instrumentation installed in the dam indicate that, there is no unexpected leakage phenomenon in this dam.

Dam safety monitoring is an essential task in risk management and normal operation of these infrastructures. A proper modeling and analysis of the structural responses under the given loads and ambient conditions, and verifying the measured data with the simulated ones are all essential tasks in dam safety monitoring. Seepage control is one of these tasks in dam surveillance, especially for earth dam [14]. Wang et al. [15] established a monitoring model to analyze the long-term trend and short-term fluctuation of a gravity dam seepage behavior, where the support vector machine is used as a baseline model for prediction. Guo et al. [16] presents an analytical for the prediction of pore water pressure inside embankment dams.

Taham dam was numerically modeled with Plaxis software [17]. The values obtained from the modeling in the pore water pressure parameter were in good agreement, but slight differences were observed for stress.

Compared the results of instrumentation and numerical analyses in the Siah Sang embankment dam, which found a good agreement from the comparison between the results of numerical analysis with the data obtained from the instrumentation [18].

The use of back analysis in Geotechnical engineering has been significantly expanded and has been widely recognized as a powerful tool for evaluating field-measurement information. One of the reasons behind the popularity of back analysis, among engineers is the lack of laboratory restriction and inside tests as well as a complete description of soil profiles [19]. In 2008, Gikas and Sakellariou made a comparison between the changed horizontal shapes obtained from the monitoring of the dam and the back numerical analysis of the earth dam. Modeling of this dam is done using Z_soil software and finite element method. Movements were measured at various stages of construction and exploitation of the dam. The settlement behavior of the Shuibuya dam at the time of construction, initial impounding and two years after exploitation were investigated. They performed a two-dimensional numerical analysis using Finite Element Method (FEM) and comparison of the results of the measured settlement by the instrumentation. Furthermore, they performed the back analysis using Hybrid General Algorithms (HGAs) [20]. The results demonstrated that, this is a successful method for controlling dam deformation. In addition, the results of settlement showed that after the first impounding, the reservoir increased, but the value
of settlement decreased and tended to be fixed over time [21]. From back analysis in Geotechnical projects, studying the type of stability of slopes and correction methods of instability can be mentioned [22], analyzing types of tunnel, caverns and galleries stability [19], and also the calibration of models [23], analyzing, and estimation of probability of fracture expansion and its effect on the safety of concrete dams [24], analyzing earth and rock fill dams during all stages of construction and completion of construction [25]. The use of back analysis as a suitable method for selection of foundation and loan sources [26]. Rashidi and Haeri investigated the method of back analysis performed on the Gavoshan dam in Kurdistan and obtained actual parameters at the end of construction [27]. In 2017, Wen et al. evaluated measurement results that were obtained from a detailed deformation-monitoring system, and numerical analyses and measurements were compared and geodetic measurement results were confirmed with numerical analyses [28]. Pramthawee et al. performed the time dependent analyses of a high rockfill dam. 3D finite-element analyses without creep and with creep considerations of a rockfill dam were compared with the in situ measurements. In addition, measurement results were verified by numerical analyses [29].

Evaluation and control of the stability of the earthly dams due to the possibility of occurrence of phenomena such as settlement, hydraulic failure, and leakage and pore water pressure are important. This is done by monitoring. Dam's monitoring is a method for investigation of dam behavior in the different state of construction, impounding and exploitation period. Numerical modeling is one of the methods of analysis and control of earthly dams. Numerical modeling is one of the methods for analysis and control of earth and rock fill dams. Comparing the monitoring and numerical modeling can help to analyze the stability of these dams. By comparing such analysis with reality, can be found that they are the same instrumentation results, in addition to validating the numerical analysis, which are the basis for performing the back analysis to obtain precise Geotechnical parameters. Accordingly, extracting final parameters using back analysis method of earth rock fill dam to predict dam performance is the main purpose of this article. Therefore, according to what was said, analysis of the stability and monitoring of earth and rock fill dams is important and necessary and so far, no research has been done on Lurestan Rudbar dam, so this article addresses this issue.

2. Main Features of Rudbar Dam
Rudbar Dam, 90 km from the city of Aligudarz in Lurestan Province, Iran is located on the Dez River Fig. 1. The main purposes of building this dam are providing the agricultural water, producing electricity-watery energy (986 GW/year), reducing the cost of thermal power depreciation and control of floodwater. To specify the behavior of the Rudbar Dam, different types of instrumentation including piezometers, measuring cells of the total pressure of the soil mass and Inclinometers were installed in three sections the Rudbar Lurestan dam body of consists of several parts including clay core, filter and transition zone, pebble shell and upstream body slope protective layer (Rip Rap). The dam core consists of GC material with a maximum particle size of 75 mm and filter material, including fine-grained part with a maximum size of 20 mm and coarse-grained part with a maximum size of 50 mm. Also, the particle size of the transition zone or drain and the dam crust are 20 and 120 cm, respectively.
The core of the dam is protected by two-layer filters and a transitive drain in upstream and downstream of the dam. The crust of the dam is divided into two parts, one part is transitive crust, or drain, and the other part is a rockfill crust. Transitional crust is between the filter and shell rockfill. The outer part of the rockfill uppermost crust is also protected by a beaching layer. With respect to earth height, type of foundation, and depth of the valley, the most installed instrumentation is in the C5-5 section, which is the most important and critical dam section in terms of maximum settlement, pore water pressure and total stress. The critical section, different levels as well as zones of different parts in Fig. 2. As shown in Fig. 3, construction of the Rudbar dam began in October 2011, and its last layer of the embankment (for 24 months) finished in October 2017, and also initial impounding of the Rudbar dam began in June and lasted until ten months, April 2017. In the following, the technical characteristics of the Rudbar dam are given in Table (1).

3. Properties of Materials and Numerical Modeling

Rudbar dam was modeled using FLAC 2D software based on the difference method. Mohr-Coulomb elastoplastic model is a behavioral model used for the core, filters, upstream and downstream.

3.1 Properties of Materials, Material properties used in numerical modeling have been extracted from laboratory experiments of soil mechanic in a fine and coarse material of the dam. The initial values of the Mohr-Coulomb model parameters, which include Elastic Modulus (E), Poisson ratio (\(\nu\)), friction of soil (C), friction angle (\(\phi\)), dilation angle and stress (\(\psi\)), hydraulic conductivity (k) have been presented in Table 2, these values have been used for initial calculations and inputs to the software.

3.2 Back Analysis

An alternative technique is to employ the machine learning methods and data-mining. Having a great capability to handle the nonlinear database, the machine learning methods have been proved to be powerful tools in dam engineering. Inverse analysis is one of these methods [30, 31, and 32]. Back analysis is an effective way to investigate and modify soil parameters. In addition, the results of the back analysis can be useful for project evaluation, and designs that are similar in structure can also be improved [33]. Back analysis of data from instrumentation and monitoring of dams at the time of construction and operation, in addition to almost surly evaluation of Geotechnical parameters, determines the stability status of the structure. In general, back analysis is a method that predicts the parameters governing a system by analyzing the output behavior of that system. Overall, the methods of back analyses are classified in two inverse and direct methods [34].

Generally, performing a back analysis requires two parts. The first part includes the stress analysis method using numerical methods to determine the stress, strain and displacement distribution for the problem in question; and the second part contains an appropriate algorithm of optimization that minimizes the difference between the values measured in situ and the data obtained from the stress analysis [35]. This difference is expressed in the form of an error function. In this paper, a direct displacement-based method has been used. One of the great advantages of the direct displacement method is its easy application in nonlinear problems and elastoplastic materials. Although this
method requires more time for computation and still requires a great deal of repetition in computing, nowadays, it is widely used in Geotechnical engineering [36]. In general, there are three different algorithms for direct back analysis: the invert method, multivariate method, and the alternating univariate methods [37]. In the univariate method, we change only one parameter at each step and the other parameters are constant. After optimizing one parameter, in the next step, we change another parameter and keep other parameters constant. We continue this until optimal values of all parameters are obtained. In the multivariate method, unlike the invert method, parameter optimization is performed simultaneously. So the model with the least error is considered as (the optimal model) and the parameters corresponding to it are called (the optimal parameters). The alternating univariate method is an extended form of the univariate method that improves it. In this method, after finding the optimal values of the parameters according to the invert method, in the next step all the parameters are simultaneously changed. Simultaneous change of parameters continues until the target function reaches the desired value.

In other words, the description of the back analysis is that the initial parameters were obtained when soil mechanics experiments were performed on the borrow materials. These materials are transported to the site of the dam, crushed into layers with a certain thickness and reach a certain density. For this reason, it is necessary to obtain the actual parameters of the materials when they are in the dam body by means of instrumentation and back analysis results. So, the most main input parameters to the software must be changed according to the actual dam behavior obtained from the information of instrumentation, to the extent that the actual behavior of the dam is created in the model. The solution process algorithm is illustrated in Fig. 4.

The displacement-based direct back analysis method was used for back analysis of the Rudbar dam and finding the optimal material parameters because of the availability of the displacement values measured by the tube of the deflectometer installed in the dam body. This method is based on the optimization of the mechanical parameters of the materials by the trial and error method. For this purpose, the error function (Eq. 1) was written in the FISH programming language in order to minimize the difference between the displacements measured by the deflectometers tubes and the displacements calculated by numerical modeling.

\[ \varepsilon(P) = \sqrt{\frac{1}{n} \sum_{i=1}^{n} \left( \frac{u_i^m(p) - u_i}{u_i} \right)^2} \]

(1)

Where \( n \) is number of measurement points, \( i = 1, 2, \ldots, n \); and \( u_i \) and \( u_i^m(p) \) are the measured and calculated values of the displacements, respectively, through numerical analysis at the corresponding points. The value of \( u_i(p) \) depends on the unknown parameters of the model that are aggregated in the vector \( P \). Before performing a back analysis, it is better that the results of the instrumentation to be processed and the incorrect displacements caused by error of reading or
improper performance of instrumentation to be removed. In conventional analysis, a mechanical model capable of introducing the mechanical behavior of a structure is assumed. After assuming this mechanical model, the values of mechanical constants are determined using field and laboratory experiments. According to issues regarding relationship between the values of the back analysis, the field measurements (instrumentation), and the methods of design and execution, it can be concluded that the back analysis is completely different from the conventional analysis.

It is assumed that, a typical analysis of a mechanical model is capable of introducing the mechanical behavior of a structure. After assuming this mechanical model, the values of mechanical constants are determined using field experiments and laboratory tests. According to Fig. 5, it can be seen that the input data of the typical analysis are used to calculate settlement values, pore water pressure, and stress.

### 3.3 Numerical Analysis and Modeling

The Rudbar dam was modeled using FLAC 2D software, which is based on the finite difference method. Regarding this research, the Mohr-Coulomb elastoplastic behavioral model was used for the materials used in the body. The Mohr-Coulomb model is one of the most well-known soil behavioral models, and because in this model, there are most of the basic parameters of soil such as plastic and elastic, it is suitable for modeling most soil behavioral modes. This model is used in many researches due to its simplicity and no need for multiple parameters.

The parameters used in this behavioral model are cohesion (C), friction angle ($\phi$), Elastic Modulus (E), Poisson's ratio ($\nu$), unit weight ($\gamma$) and coefficient of permeability (k). Table 2 represents initial values of the mentioned parameters.

There is a criterion in the Mohr-Coulomb behavioral model that defines the boundary between elastic and plastic deformations and the behavior of materials after entering the plastic zone is quite distinct from the behavior before crossing this boundary. In this behavioral model, the failure envelope is obtained by the well-known Mohr-Coulomb criterion, which is a function of shear failure ($f_s$), considering the tensile failure criterion ($f_t$). The failure level of $f_s$ and $f_t$ is obtained from the equations 2, 3 and 4:

\[
(2) \quad f_s = \sigma_1 - \sigma_3 N_\theta + 2C\sqrt{N_\theta}
\]
\[
(3) \quad f_t = \sigma_1 - \sigma_3
\]

Where $N_\theta$ in this equation:

\[
(4) \quad N_\theta = \frac{1+\sin\theta}{1-\sin\theta}
\]
Where in the above equations, $C$ is adhesion, $\phi$ is angle of internal friction, and $\sigma_1$ and $\sigma_3$ are tensile strength.

The parameters of the Mohr-Coulomb behavioral model include $\phi$ and $C$, which are related to the plastic mechanism of the model and $E$ and $\nu$ parameters, which are related to the elastic mechanism of the model. These parameters are determined by ordinary and common experiments like triaxial tests. By simulating a drained triaxial test with the Mohr-Coulomb model, it can be found that the behavior presented by the model before the failure is controlled by the $E$ and $\nu$ parameters, and the maximum shear strength is controlled by $\phi$ and $C$ parameters.

The Rudbar dam was modeled in 31 layers that the height of each layer was 5 m. An element grid $97 \times 31$ has been used in modeling the dam body. In the first step, quadrilateral elements with four nodes with a length to width ratio of approximately 1 are considered in producing the mesh. After designing total grid of the dam body, extra elements were removed and optimal mesh model of critical section was obtained according to Fig. 6. In the second step, after creating the dam model geometry, the boundary and initial conditions are applied to the model so that lateral boundaries can be displaced and at the lower boundary, horizontal and vertical displacements are assumed to be zero.

In the third step, for the bedded modeling and investigation of the consolidation phenomenon and the pore water pressure on the dam, at first, the timing plan of the dam construction was determined based on the actual time of dam body embankment that took more than 1440 days. Thus, the total construction time was given to the software for each of the 31 layers of the body. Then, the first layer of the dam body was created, and by performing mechanical computational steps, the unbalanced force reached zero. In the following, all the layers of the dam body were constructed, thus step construction of the dam in the model software and then impoundment of the Rudbar dam at seven stages that the height of each stage was 20 meters (which lasted for 10 months) were modeled. The maximum water stage in the reservoir has been recorded 141 m. To simulate impoundment of the dam, the mechanical force due to the weight of the reservoir water was applied to the upstream crust, then the corresponding hydrostatic pressure of the reservoir water was applied in the same area and the model was run until the allotted time.

It should be noted that due to the very low permeability of the rock bed and construction of a concrete slab under the core as well as the partial settlement at the time of construction (7 cm), the modeling of foundation was ignored.

4. Results and Discussion
In this part, at first, the contours obtained from the modeling of pore water pressure, vertical and horizontal stress at the end of construction and impounding are presented in Figures (7-12).
4.1. Pore Water Pressure
Concerning embankment dams, the monitoring of pore water pressure is crucial because it is the main indicator of internal erosion and seepage problems and has a significant role in the stability of geotechnical works [38].
Installed piezometers in at the core of the Rudbar dam were placed in 6 different levels. This paper focuses on the most acute pore water pressure or the most pore water pressure recorded by piezometers in the core of the dam located at 1615 m. The electrical piezometers installed on this level consist of 7 piezometers that piezometer EP16 and EP19 are obstructed and their results are not available. Fig. 13 (a) shows the pore water pressure diagram of electric piezometers at a level of 1615 m. As it is shown, as the height of the dam increases, the pore water pressure also increases. Another significant note is that at the time of construction, the middle piezometers of the core dam have recorded more water pressure compared to lateral piezometers and it's because of more water pressure of middle of the core than lateral of the core.

4.2 Comparison of Pore Water Pressure Measured by Instrumentation with a Numerical Model
In this study, because of the highest pore water pressure recorded at the bottom of the core, i.e., Level 1615 m is recorded by the instrumentation, so the comparison between instrumentation results and the numerical modeling is done at this level. Comparison of the pore water pressure measured by the instrumentation and the numerical model at the end of the dam body construction is shown in Fig. 13 (b).
In the figure 13 (b), the maximum pore water pressure of instrumentation is obtained 736 kPa and 780 kPa at the end of construction and in the numerical modeling, respectively. It is observed that there is good agreement between the results. The numerical analysis and instrumentation suggest that the maximum of pore water pressure occurred at the center of the core and the minimum of pressure during construction and end of construction is recorded in upstream and downstream of the core.

4.3. Total Vertical Stress
Manometers that measure total stress (vertical and horizontal) were installed at the core of the dam in five different levels with three clusters that measure the soil pressure in three directions: vertical, horizontal and in accordance with the core axis. This study focuses on the most critical level in the dam body, the 1615 m level where PC1, PC2 and PC3 pressure cells are installed on this level and the following measurements of these pressure cells are presented. Pressure cells, which have been horizontally installed, measure stress in the vertical direction ($\sigma_y$), pressure cells that are parallel to the dam axis, measure stress along the X-axis ($\sigma_x$), and pressure cells, which have been vertically installed on the dam axis, measure stress along the Z-axis ($\sigma_z$).
Fig. 14 (a), shows measurements of total vertical stress in the core of the dam at the end of construction in different embankment levels. As can be seen, the vertical stress at the center of the
core is higher than the lateral side and by increasing the height of the dam embankment, vertical stress has been increased.

Fig. 14 (b), shows changes of pressure cells at the end of construction and impounding at level 1615m.

**4.4. Comparison of the Total Vertical Stress Measured by the Instrumentation with the Numerical Model**

Considering that highest vertical stress is recorded at the bottom of the core, i.e., the level 1615m by the instrumentation; so, the comparison between the instrumentation results and numerical model is performed at this level. Fig. 15 (a), shows the verification of vertical stress measured by the instrumentation and the numerical model at the end of construction. The maximum vertical stress of instrumentation at the end of construction is obtained 2530 kPa and in the numerical model is obtained 2442 kPa. By observing figure 15 (a), it can be said that, there is a good convergence and consequently the maximum vertical stress occurred at the center of the core as well as the results of the numerical model and instrumentation indicated that the least stress occurred in the upstream of the core during construction and downstream of the core occurred at the end of construction.

In this article, the vertical stress of impounding stage is presented. Fig. 15 (b), shows a comparison of the results of instrumentation and the numerical model. Fig. 16 illustrates vertical stress variations of PC1, PC2 and PC3 cell pressure derived from a numerical model and instrumentation during construction and impounding.

As shown in figure 16 (a), there is a little difference between the instrumentation and the numerical model, because there is less density around the cell than other parts of the core to prevent damage; so rigidity of the other parts is greater than surrounding of the cell, but changes in cell pressure PC2 and PC3 are in good agreement as shown.

**4.5. Evaluation of Arching Ratio**

Whatever size of the arching ratio is being larger, the occurrence of the arching is less in that location, and whatever size of the arching ratio is less, the occurrence of the arching is more in that location. Increasing the amount of arching ratio decreases the soil core pressure from the hydrostatic pressure, resulting in cracks in the core. This phenomenon is known as the failure, hydraulic [39]. Eq.7 shows the relationship of the arching ratio:

\[
Ar = \frac{r_u}{R_u} = \frac{\sigma_{pc}}{\gamma h}
\]

\(\gamma\) is unit weight column of soil above the instrumentation. \(h\) is the height of the soil column above the instrumentation. \(\sigma_{pc}\) is total stress measured by instrumentation.

In Fig. 17 (a), the arching ratio at the end of construction and impounding is shown at level 1615 m. As it is known, at the end of the construction, the lowest arching ratio was recorded 0.53 at the
upstream of the core, but at the end of the initial impounding, the highest increase was also observed in upstream of the core, and this shows the effect of impounding on the arching ratio.

4.6. Comparison of Arching Ratio of the Instrumentation with the Numerical Model
The comparison of arching ratio in Fig. 17 (b), shows that the maximum and minimum of arching ratio obtained from numerical modeling are 0.68 and 0.59, respectively.

After initial impounding, it should be investigated whether the hydraulic failure phenomenon occurred in the dam body or not. One solution is to obtain the arching ratio. After impounding, due to the water pressure on the upstream of the dam, the arching ratio must be checked, because if the amount of this ratio is low at the end of the construction, the possibility of the arching is very high. Of course, in Rudbar dam, the lowest amount of arching ratio was 0.53 at the end of construction, which reached 0.59 in impounding, which not only declined but also increased.

4.9. Settlement
At Rudbar dam, three Inclinometers (settlement tubes) are installed in the upstream center of the core and downstream, which measure the settlement in these sections. As shown in Fig. 18, shows the settlement changes in the core (IN2) at nine intervals and in different heights at the time of construction and initial impounding of the dam. According to the announcement, construction of the dam was completed in October 2012; the maximum settlement was 137.5 cm and happened in the core of the dam in this period. The initial impounding began in March 2016, also, in this period; maximum settlement of the dam was recorded 156.9 cm on the inclinometers critical sections.

4.10. Comparison of Settlements Obtained from Instrumentation Measurements and Numerical Model
One of the parameters for stability analysis of earth and rockfill dams is the most of settlement experience of earth rockfill dams during construction [24]. Fig. 19 shows a comparison of the settlement in the center of the core at the end of construction (a) and initial impounding (b). The maximum settlement recorded by the instrumentation and modeling at the end of the construction was 137.5 cm and 146/5 cm, respectively, as well as after the impounding in instrumentation and modeling was obtained 156.9 cm and 147.8 cm, respectively.

4.11. Results of Back Analysis
Regarding to verifications of parameters such as pore water pressure, vertical and horizontal stress, the actual Geotechnical parameters of the dam body were obtained using the back analysis method. Table 3 presents the values of the final parameters obtained from the back analysis.

4.12. Comparison of Actual Geotechnical Parameters Obtained from Back Analysis with Initial Parameters
The final parameters obtained by the back analysis were given in Table 3, and some parameters were presented as a bar graph in this paper. Fig 20 (a) shows the Poisson ratio Fig. 20 (b) shows the friction angle in different conditions of the dam. Part (a) of the core, filter 3 and drain have increased in the final condition, and in the other parts, there was no change, but in part (b) all of the dam sections changed in the final condition.
Fig. 20 (c) shows dry unit weight of the core, which is constant and all parts are increased in the final condition.

5. Conclusion
The purposes of this article include providing a general assessment of performance of the dam at the time of construction and the initial impounding using the data obtained by the instrumentation of the Rudbar rockfill earthen dam, and also reaching the actual Geotechnical parameters in the body of the Rudbar dam at the end of the construction by back analysis method.

1. The highest settlement rate of the Rudbar dam was at the core and at a height of 85 m from the floor. Rudbar dam experienced 87% of its total settlement construction at the end of construction. Also, a settlement maximum of the Rudbar dam at the end of construction is 1.37 m, which is less than 1% of the dam height. In comparison with a settlement in the other dams such as the Alavian dam with a height of 80 m and a settlement of 1.30 m (about 1.6% of the body height), Gavoshan dam with a height of 123 m and a settlement of 2.7 m (about 2.1% of the body height settlement of the Rudbar dam is acceptable.

2. The maximum pore water pressure was at the end of the construction at the center of the core and the lateral part of the core was less tolerated; but in the impounding stage, maximum pore water pressure was recorded in upstream of the dam; so one of the main reasons is being near this part of the upstream of the core.

3. The assumptions made by the dam are constructed in 31 layers and the impounding, which occurs in seven states, is appropriate. Specifically, the assumption of fewer layers at the time of construction would change the results of pore water pressure due to the thick layers were placed and their abruptly weight addition. Placing layers with the thickness more than 8 m causes extra pore water pressure in the initial stage of construction, and it is better to assume the number of layers in a way that their thickness being 5 m for analyzing earth and rockfill dams.

4. Arching ratio is one of the criteria for controlling hydraulic failure in earth rockfill dams. The arching ratio of the critical level of the Rudbar dam body is 0.53 - 1.6. This amount has good agreement in comparison to arching ratio occurred in large dams such as Asvatvan dam in Norway with an arching ratio of 0.33 to 0.9 and Karkheh dam with an arching ratio of 0.47 to 0.75 and Ardak dam with an arching ratio of 0.63. After impounding of Rudbar dam, the arching ratio rate increased and of arching decreased.

5. According to the results, it can be said that the back analysis method is a suitable method for obtaining actual parameters in earth and rockfill dams.

6. Based on the results of evaluations of instrumentation system of the Rudbar dam body, it can be said that instrumentation system and the measuring equipment of this dam quantitatively and qualitatively have sufficient health to continue the measurement and recording the instrumentation data.
7. In the values obtained from the back analysis, the most change has been observed in the dry unit weight parameter and also all the parameters have experienced a constant or increasing trend. Generally, it can be said that when the back analysis was not done on the geotechnical parameters of the dam, there was no mentality about the actual geotechnical parameters in the dam body, but a proper view of the actual geotechnical parameters in the body of Rudbar dam was obtained by performing this paper.

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Fig. 1. Location and view of the body of Rudbar dam.
Fig. 2. The critical section, different levels as well as zones of different parts of Rudbar Dam.

Table 1. Technical characteristics of the Rudbar dam and its reservoir [25].

| Dam detail                           | Value          |
|--------------------------------------|----------------|
| Height from bed to crest Largest cross-section | 153m           |
| Width of the dam crest               | 15m            |
| Length of the dam crest              | 185m           |
| Dam crest elevation a.s.l            | 1765m          |
| Normal water level a.s.l             | 1756m          |
| Area of reservoir length             | 20 km          |
| Dam body material volume             | $4.596 \times 10^6$ m$^3$ |
| Volume reservoir                     | $228 \times 10^6$ m$^3$ |

Table 2. List of properties of materials for numerical analysis (primary parameters) [25]

Fig. 3. Construction and impounding periods of Rudbar dam.
### Table: Material descriptions

| Material descriptions | k (m/s) | $\phi^\circ$ (kPa) | C (kPa) | $kN/m^3$ ($\gamma$) | E (MPa) | $\psi^\circ$ | $\nu$ |
|-----------------------|--------|---------------------|--------|---------------------|---------|-------------|------|
| Core                  | $1 \times 10^{-1}$ | 25 | 50 | 23.4 | 21.5 | 35 | 0 | 0.35 |
| Filter 3              | $5 \times 10^{-1}$ | 39 | 0 | 21.9 | 19 | 70 | 7-10 | 0.33 |
| Filter 2              | $5 \times 10^{-2}$ | 39 | 0 | 22.2 | 19.5 | 70 | 7-10 | 0.30 |
| Drain                 | 1 | 45 | 0 | 23.2 | 21 | 70 | 7-10 | 0.25 |
| Shell Upstream        | $1 \times 10^{-1}$ | 45 | 0 | 23.5 | 21.5 | 70 | 3-5 | 0.25 |
| Downstream            | $1 \times 10^{-1}$ | 45 | 0 | 23.2 | 21 | 70 | 3-5 | 0.25 |

### Diagram:

**Start**

- **Model Setup:**
  (1) Model geometry; (2) Describe constitutive behavior and material properties; and (3) Define boundary and initial conditions

- **Step to obtain equilibrium**
- **Monitoring measurements**

- **Back analysis**

  Are measured and computed values in good agreement with each other with low error?

  - **Yes**
    - Assigning geotechnical parameters, stress ratio, and joint parameters
    - **Model running natural condition**
    - **Finish**
  - **No**

**Fig. 4.** Solution algorithms for back analysis.
**Fig. 5.** Typical analysis and back analysis.

**Fig. 6.** Mesh generation of Rudbar dam body at the critical cross-section.

**Fig. 7.** Pore water pressure at the end of construction (body height 155 m) (unit: Pa).
Fig. 8. Pore water pressure at steady-state (unit: Pa).

Fig. 9. Vertical stress construction at 155m height (unit: Pa).

Fig. 10. Vertical stress of initial impounding to normal levels (unit: Pa).

Fig. 11. Settlement construction at 155m height (unit: m).
Fig. 12. Settlement initial impounding to normal levels (unit: m).

Fig. 13. Pore water pressure changes in the time of construction (a) and (b), comparison of the pore water pressure measured by instrumentation and numerical model at the end of construction at the level 1615m.

Fig. 14. Total vertical stress of core dam at the time of construction (a) and (b), changes of the total vertical stress at level 1615m.
Fig. 15. Comparing total vertical stress measured by the instrumentation and numerical modeling at the end of construction (a) and at the end of impounding (b), at level 1615 m.

Fig. 16. Changes of total vertical stress measured by instrumentation and numerical model in pressure cells PC1, PC2, PC3 in level 1615 m.
Fig. 17. The arching ratio of core of the dam at the end of construction and initial impounding and comparing arching ratio measured by instrumentation and numerical model at the end of construction at level 1615 m.

Fig. 18. Core settlement obtained from IN2 along with dam height at different times.
Fig. 19. Comparison of the core settlement obtained from instrumentation and numerical model at the end of construction and initial impounding.

Table 3. Final parameters in the back analysis (actual parameters)

| Material descriptions | $k$ (Cm/s) | $\phi^o$ (kPa) | C (kPa) | kN/m$^2$ | $E$ (MPa) | $\psi$ | $\nu$ |
|-----------------------|------------|----------------|---------|----------|-----------|--------|-------|
| Core                  | $1 \times 10^{-1}$ | 28      | 50      | 23.5     | 21.5      | 43     | 0     | 0.37  |
| Filter 3              | $5 \times 10^{-10}$ | 40      | 0       | 21.9     | 20        | 70     | 9     | 0.37  |
| Filter 2              | $5 \times 10^{-4}$  | 40      | 0       | 22.2     | 20.5      | 70     | 9     | 0.30  |
| Drain                 | $1 \times 10^{-3}$  | 46      | 0       | 23.2     | 22        | 70     | 9     | 0.37  |
| Upstream              | $1 \times 10^{-1}$  | 46      | 0       | 23.5     | 22.5      | 70     | 5     | 0.25  |
| Downstream            | $1 \times 10^{-11}$ | 46      | 0       | 23.2     | 22        | 70     | 5     | 0.25  |
Fig. 20. Comparison of Poisson's ratio, friction angle and dry unit weight in the initial conditions obtained from the back analysis (Final).
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