# Structural Analysis Framework for Concrete-Faced Rockfill Dams

Mehdi Modares, M.ASCE[1]; and Juan E. Quiroz, M.ASCE[2]

**Abstract:** Concrete-faced rockfill dams (CFRDs) are commonly built around the world. As energy demands soar and construction methods evolve, the interest in the construction of CFRDs with larger heights has increased tremendously. However, because of the construction of higher CFRDs, some dams have experienced considerable fractures at the concrete faces. Well-known cases include Campos Novos (Brazil), Barra Grande (Brazil), and Mohale (South Africa), where in some instances these cracks have led to dewatering of the reservoirs to allow for the concrete slabs' repairs. The development of these fractures may be attributed to the highly deformable rockfill body. In general, the state-of-the-art design of CFRDs is mostly based on common practice rather than rigorous analysis procedures. And as such, cracking problems because of deformability of the rockfill may not be properly predicted unless a detailed analysis is performed. In this work, a new framework for the analysis of CFRDs is developed that is capable of predicting the possible failure of a concrete facing. As part of this framework, a comprehensive nonlinear finite-element analysis scheme is developed to model the construction sequence, the contact interaction between the concrete facing and the rockfill body, and the impounding of the reservoir. As a case study, using the developed framework, the Kárahnjúkar CFRD (Iceland, 198 m in height) is analyzed, the results are validated by the field measurements, and suggestions for mitigation measures are provided. This methodology, based on the results of the investigation, provides guidelines and establishes a framework for analysis of CFRDs that can be used for design purposes. **DOI:** 10.1061/(ASCE)GM.1943-5622.0000478.


**Author keywords:** Dam; Concrete-faced rockfill; Finite-element analysis.

## Introduction

Concrete-faced rockfill dams (CFRDs) are one type of embankment dams built with compacted rockfill in layers or lifts and covered with concrete slabs at the upstream face as part of an impermeable barrier for the water. The concrete faces are jointed to the sound foundation through a reinforced concrete section known as a plinth. The water barrier is connected to the foundation at the toe slab with an appropriate foundation treatment at the upstream and downstream locations. These dams are more commonly built on stiff foundations, but some CFRDs are built on weathered rocks, and alluvial foundations could have the potential for a piping mode of failure.

The body of the dam is usually divided into zones designated with numbers and letters depending on the particle size, material type, and purpose. A typical CFRD zoning is shown in Fig. 1 (Cooke and Sherard 1987).

Zones 1A and 1B protect the upstream concrete faces, and are usually cohesionless silt or fine sand. Zones 2A and 2B support the concrete faces and are made of processed granular materials. Zones 3, 4, etc. are quarry rockfill zones. Zone 3A limits the void size. Zone 3B resists water pressure and controls face deflection. Zone 3C is composed by larger rocks and settles the most during construction. Additional zoning can be defined as required.

As construction methods evolve and inevitable energy demands soar, the size of hydropower projects increases. This boost translates into higher dam sizes needed to achieve greater heads and sufficient reservoir capacities. Compared with other types of dams, CFRDs are straightforward to construct, economical, and generally adaptable to terrain geometry; materials are usually available in close proximity. However, some high CFRDs, such as Campos Novos (Brazil, 202 m height), Barra Grande (Brazil, 185 m height), Mohale (South Africa, 145 m height), Aguamilpa (Mexico, 187 m height), and Tianshengqiao (178 m, China), have experienced significant structural failures because of concrete-slab fractures, which cause considerable leakage (Ma and Cao 2007).

The water-retaining membrane system of CFRDs is mainly dependent on the integrity of the concrete slabs because they act as an impervious membrane at the upstream face of the dam. For this reason, cracking of the concrete slabs permits the water to infiltrate the rockfill, thus deteriorating the inside material. The repair demands for these cracks lead to reservoir dewatering and considerable cuts in power revenues. Other authors have focused on the onset of failure in rockfill dams due to mass sliding, including the flow of water within the dam body (Larese et al. 2013). More recently, long-term creep settlement effects have been studied using viscoplastic theories (Dolezalova and Hladik 2011).

The development of these fractures is attributed to the highly deformable rockfill body. The rockfill's flexibility compared with the stiffness of the concrete membrane directly affects the behavior of facing slabs. As the entire dam body deforms, the concrete slabs follow this deformation, resulting in excessive stresses within the concrete slabs. Moreover, during impoundment, the pressure on the slabs increases the shear transfer between the concrete and the rockfill below it, inducing additional stresses.

[1]Assistant Professor, Dept. of Civil, Architectural and Environmental Engineering, Illinois Institute of Technology, 3201 S Dearborn St., Chicago, IL, 60616 (corresponding author). E-mail: mmodares@iit.edu

[2]Adjunct Professor, Dept. of Civil, Architectural and Environmental Engineering, Illinois Institute of Technology, Chicago, IL, 60616; Lead Structural Engineer, MWH Inc., 175 W. Jackson Blvd., Chicago, IL, 60604.

Note. This manuscript was submitted on August 1, 2013; approved on November 25, 2014; published online on May 12, 2015. Discussion period open until October 12, 2015; separate discussions must be submitted for individual papers. This paper is part of the ***International Journal of Geomechanics***, © ASCE, ISSN 1532-3641/04015024(14)/$25.00.

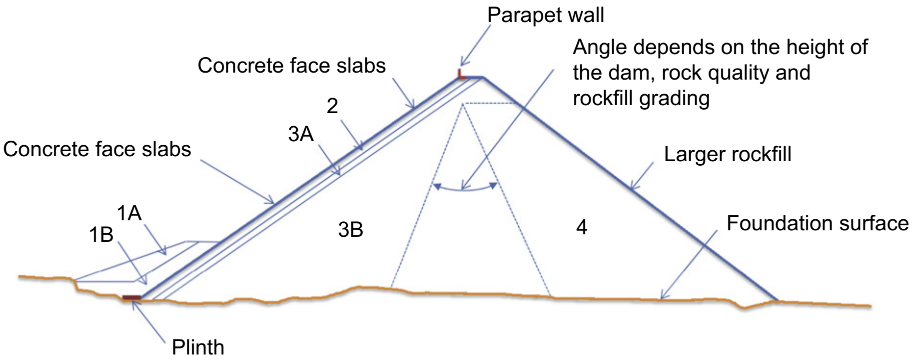



**Fig. 1.** Typical CFRD configuration



Despite the popularity of CFRDs, designs are mostly based on common practice rather than rigorous analysis procedures (Cooke 1984). However, owing to the experienced structural failures of CFRDs, a more comprehensive methodology for analysis and design is needed. Furthermore, because of site conditions or unexpected situations, numerous design changes and mitigation measures are required while construction is in progress. These changes and mitigation measures in design require structural analyses for estimating and comparing their effectiveness.

In this work, a new analysis framework for CFRDs is developed. This framework introduces a comprehensive nonlinear finite-element analysis (FEA) scheme for CFRDs to model the construction sequence, the contact interaction between the concrete facing and the rockfill body, and the impounding of the reservoir.

As an example for the method developed in this research, the Kárahnjúkar CFRD is analyzed. This dam is the tallest in Europe with a height of 198 m, a length of about 730 m, and an installed capacity of 690 MW (Anon 2009 and Johannesson 2006). The analysis includes the staged construction (resulting in an updated and larger stiffness matrix at every step of the analysis), as well as the contact interfaces between slabs, slab/rockfill, and upstream-backfill/slabs. In addition, the reservoir impoundment is modeled in stages and correlated with recorded data from instrumentation. All of the interfaces include a normal and tangential behavior allowing contact, separation, and slippage between the different surfaces. The data collected during construction and reservoir impoundment is used for calibration of the computational model. Then, suggestions for mitigation measures are provided. The method developed, as well as the suggested mitigation measures, can be followed in other similar cases and establishes an analysis framework for these types of dams.

In the following sections, the current challenges of CFRD analysis and design are discussed first. Then, the new framework for analysis of new and during-construction CFRDs is presented. Next, as a studied case, a description and analysis of the Kárahnjúkar dam along with considerations determining the behavior of the structure and its instrumentation, as well as mitigation measures, are presented, followed by conclusions.

## Challenges on CFRD Design

### *Current Practice*

Rockfill deformation and permeability are the bases for the behavior of CFRDs. Rockfill is selected depending on the material availability at the dam location. Zoning and gradation are also crucial to control deformations. Moreover, well-graded and higher-density materials yield greater shear strengths. Thus, rockfill modulus estimation is essential for analysis and rockfill selection. When comparing the modulus of deformation measured during construction versus the reservoir filling modulus, some basic relationships are commonly used. The vertical modulus of deformation, $E_v$, is obtained from vertical settlement (Fitzpatrick et al. 1985) as

$$E_v = \frac{\gamma DH}{s} \tag{1}$$

where $\gamma =$ unit weight of rockfill; $D =$ depth of rockfill above settlement gauge; $H =$ height of rockfill below settlement gauge; and $s =$ settlement of the gauge (Fig. 2).

Also, some empirical approaches allow the estimation of face-slab deformations (Pinto and Marquez 1998) by using the transverse modulus of deformation, $E_t$, which is measured in the direction of the deformation (perpendicular to concrete faces) under the reservoir load as

$$E_t = \frac{\gamma_w d h_i}{\delta_s} \tag{2}$$

where $\gamma_w =$ unit weight of water; $d =$ depth of water above concrete slab; $h_i =$ height of inclined column normal to concrete slab; and $\delta_s =$ concrete slab deformation (Fig. 2).

These two moduli were defined for two phases: (1) during construction and (2) for first filling (Fitzpatrick et al. 1985). In many cases, the proposed empirical approaches relate the ratio $E_t/E_v$ with valley shape factor $(A/h^2)$, where $A =$ facing area; and $h =$ height of the dam. Furthermore, the ratio $E_t/E_v$ could produce values between 2.0 and 3.0 (Johannesson 2007). Yet, it has been suggested this ratio as a function of two different Poisson's ratios, one during construction and the other for reservoir filling.

Data from several constructed dams suggest that for narrower valleys, the measured settlements tend to decrease (Pinto and Marquez 1998). This is observed when comparing the shape factor $(A/h^2)$ with the measured vertical modulus. This result indicates the presence of a stress-arching effect across the abutments for narrow valleys, and thus emphasizes the importance of three-dimensional (3D) behavior.

The selection of face-slab thickness is usually based on previous experience rather than analytical procedures, and improvements are made depending on the dam configuration. Contraction joints are established where the slabs are expected to move toward or away from one another. These trends are usually estimated

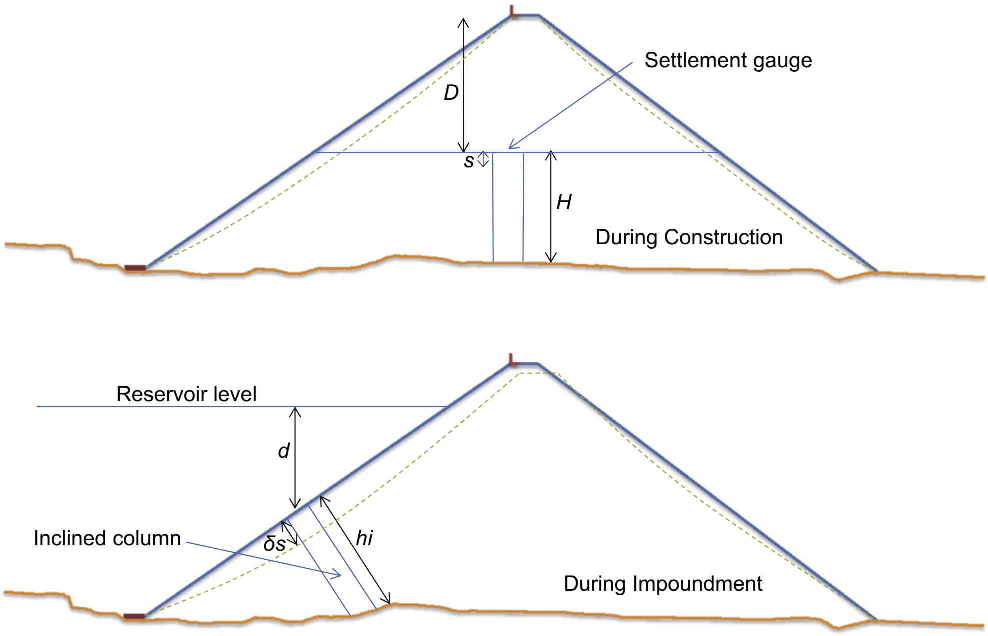



**Fig. 2.** Vertical settlements during construction and concrete face deflection

depending on the shape of the valley. The widths of the slabs are generally dictated by the equipment used for pouring the concrete.

### *Problems Encountered*

The main problem of CFRDs is cracking of the face slabs, causing leakage and leading to further damage to the rockfill body and loss of water. There are several parameters related to dam performance, such as leakage, settlements, deflections, and strain measurements. An important factor for identifying the integrity of the dam is leakage, which indicates possible damage of the perimeter joints or the face slabs. Estimation of rockfill settlements and face-slab deflections is essential in the analysis of a CFRD because these are clearly related to stresses on the concrete facing.

The behavior of concrete face slabs is directly related to the supporting zone and rockfill deformability, which depends on several aspects such as gradation, compaction, lift thickness, watering, etc. The maximum settlements are usually observed at mid-height, and the lower-third portion of the concrete facing results in a bulging deformation that induces tensile stresses on the concrete slabs (Marquez and Pinto 2005). The 3D effect of the valley permits rockfill movements towards the center of the dam that could induce additional tensile stresses of the rockfill and dragging of the slabs at the abutments (Fig. 3). The dragging effect observed on the concrete slabs, in both slope and horizontal directions, is caused mainly by the rockfill deformation. During impoundment, the normal pressure on the slabs increases friction resistance at the interface with the rockfill body, facilitating concrete membrane deformations that could lead to crack development.

### *Concrete-Slab Cracking*

On some already-built dams, there have been incidents of concrete face cracking useful for studying CFRD mechanisms related to concrete-facing cracking (Table 1).

The concrete-slab cracking observed on several CFRDs presents compressive failures, including reinforcement buckling, slab hiving, and considerable concrete spalling. This type of failure is produced when the compressive demand exceeds the capacity of the concrete slab. The design efforts are then focused on minimizing the development of these compressive stresses to mitigate the potential of cracking on the slabs. The development of the compressive stresses and failures observed is related to settlement that occurs during impoundment.

Dam designers and consultants of high CFRDs have concluded that accepted CFRD design practices used in the design of these dams do not adequately consider a number of factors that most likely contribute to the observed behavior during reservoir impounding (Schreppers and Lilliu 2009). Current practice is based mostly on engineering judgment, which can benefit greatly from a more rigorous analysis tool that provides the engineer additional information for a better design.

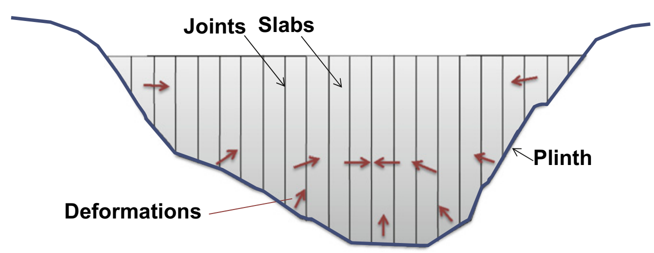



**Fig. 3.** Face slabs dragged by the rockfill deformation

**Table 1.** Precedent CFRDs with Cracking

| CFRD | Issue | Cause |
|---|---|---|
| Aguamilpa | Concrete facing cracking | Rockfill deformability |
| Tianshengqiao 1 | Horizontal cracking | Construction sequence |
| Xingó | Slabs cracking | Sharp geometry of the left abutment and Zone 3C material deformability |
| Itá | Slabs cracking | Rockfill deformability |
| Itapebi | Cracks parallel to the plinth | Foundation geometry |

### Innovation

This work proposes a novel comprehensive modeling framework with the ability to investigate different scenarios, for new designs and during-construction analyses. Using this framework, optimum scenarios are evaluated to support decisions at the design stage and to propose changes during construction, if required. Although numerous alternatives exist to alleviate a given problem during construction, in many cases, the solutions proposed might not be the most effective in terms of cost and benefit. For instance, modifying placement sequences could be more effective in some dams, or the thickening of concrete facing could be insignificant if implemented on the wrong locations, increasing constructions costs unnecessarily. The proposed analytical framework can be used for each dam individually in order to attain the best solution.

## Methodology

Fig. 4 depicts a general procedure for the analysis of CFRDs.

In numerical analyses for CFRDs, precedent experiences, and the literature must be considered for design purposes (Pinto 2009). If the dam is located on a narrow valley with a low shape factor, a 3D model must be developed to properly capture the arching effect of the stresses distributed towards the abutments. Also, the

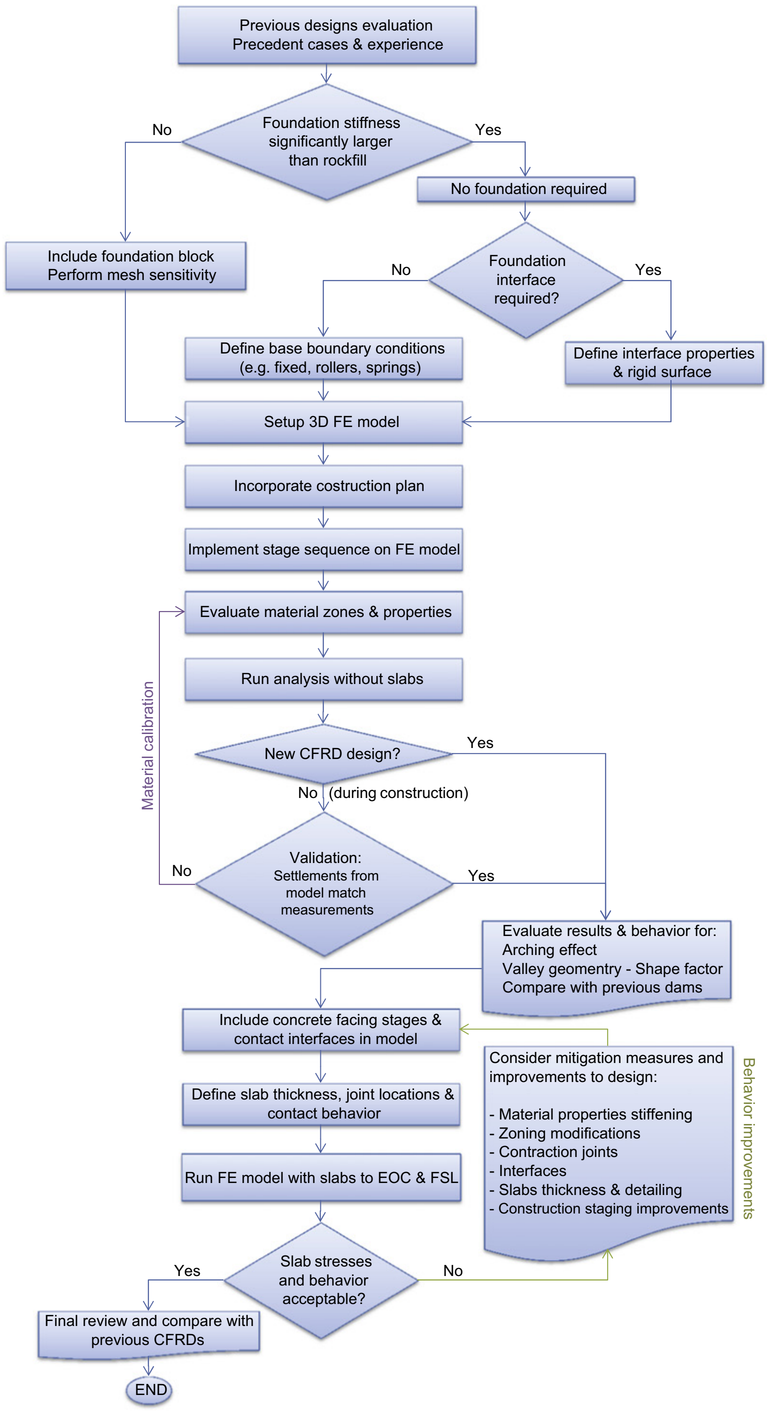



**Fig. 4.** Analysis framework for CFRDs

most relevant features must be incorporated in the finite-element (FE) model including site conditions, geometry, construction schedule, material properties distribution, canyons or marked topography, and valley shape. During the model setup, the mesh fineness must be reasonably chosen depending on model complexity and nonlinear attributes so as to achieve the most computational feasibility. The model geometry must globally reflect the main dimensions of the dam. In general, the analysis should evolve progressively from the most basic to the most detailed model.

If the foundation stiffness is considerably larger, it does not need to be included in the model, and appropriated boundary conditions must be applied at the rockfill footprint. These boundary conditions can be represented by a stiffness (spring), a rigid constraint, or a restrained boundary. The use of a proper boundary condition is judged on the basis of the material properties and geometric characteristics of the foundation footprint. Also, the foundation can be modeled as a rigid surface (without increasing the degrees of freedom) and provide contact interface properties to allow the elements to slide over the rigid foundation.

For the design of a new CFRD, construction staging is usually assumed on the basis of previous construction procedures. With a proper FE model, the designer can evaluate several scenarios in order to achieve an optimum rockfill placement in terms of deformations. Once the construction of a CFRD begins, the staging plan can be different from that assumed during the design phase because of contractor's procedures, contractual constraints, material availability, etc. Therefore, for new and during-construction CFRD projects, stage-construction analysis is a crucial procedure for obtaining realistic behavior. Material properties are usually estimated for a new dam at the design phase and depend on the site investigations. In the majority of cases, these materials change because of quarry location, process used, and further tests at the time of construction. Material properties should be updated to reflect actual during-construction conditions.

The initial FEA must be performed without considering concrete slabs to expedite the process and shorten the computational time. Using the initial FEA, the main settlements are determined and compared with measurements from instrumentation obtained from settlement cells for material calibration purposes. The main parameters used for this calibration are the moduli of elasticity and shear. These moduli are modified depending on the material zoning and location of the cells. This process is repeated until a reasonable match with the measurements is achieved. Then, the results are compared with previous dams, leading to the projection of settlements at the end of construction (EOC).

Once the model is calibrated and checked, the concrete facing is included. Similar to the rockfill staging, the concrete slabs are placed at different times, and the staging plan is updated and implemented on the model. The concrete slabs are dimensioned using the equipment for pouring, and the thicknesses are at first usually set by preceding practice. The analysis results may suggest changes to the slabs' thicknesses depending on the level of stresses. Also, slab widths may be adjusted as the construction equipment permits. Concrete-facing joints are located between slabs and can be adjusted for additional control and support. Certain forecasted deformation patterns are key for establishing the location of horizontal joints. This is first implemented by precedent cases and optimized on the basis of the analytical results. The contact behavior of the joints depends on the filler material used, if any. The filler material can be made of soft wood, Ethylene Propylene Diene/PolyMethylene fillers, or other alternatives (Pinto 2009). Later, the analysis results may suggest changes to the joints for improved behavior, such as increased spacing and material behavior selection.

Once the model is ready, improvements to its behavior can be evaluated by an iterative process. First, settlements, slab deformations, and stresses need to be within an acceptable range to prevent concrete failure. If high stresses are predicted on the concrete face slabs, they must be reduced by implementing one or several mitigation measures. The following are some of the mitigation measures:

- Produce stiffer rockfill materials (e.g., lift thickness, watering, compacting, gradation).
- Modify rockfill placement (e.g., downstream placement/staging first).
- Postpone concrete slab placement closer to the EOC phase.
- Delay concrete slab staging.
- Increase joint gaps.
- Improve filler material behavior.
- Isolate slabs from rockfill by adding bond-braker materials.
- Increase slab thickness at selected locations.

One of the main advantages of having a comprehensive model is the ability to investigate different scenarios for new designs and during-construction analyses. Using this framework, optimum scenarios are evaluated to support decisions at the design stage and to propose changes during construction, if required. Although numerous alternatives exist to alleviate a given problem during construction, in many cases, the solutions proposed might not be the most effective in terms of cost and benefit. For instance, modifying placement sequences might be more effective in some dams, or the thickening of the concrete facing will be insignificant if implemented on the wrong locations, increasing constructions costs unnecessarily. The proposed analytical framework can be used for each dam individually in order to attain the best solution.

## Numerical Example

### *Kárahnjúkar CFRD*

The Kárahnjúkar CFRD (Iceland) is one of the tallest dams of its type. The upstream section of the dam in the canyon is formed by a concrete toe wall supporting the concrete facing. The dam is located on the Jökulsá á Dal River, which originates and flows northeast from the Vatnajökull Glacier. Bedrock at the dam site consists of a series of lava flows and is primarily composed of basalt overlain by pillow lavas. At the dam site, the river is deeply incised in a canyon approximately 50–70 m wide and about 50 m deep. Above the canyon, the river valley broadens asymmetrically, with the left abutment having a flatter slope than the right.

The Kárahnjúkar CFRD impounds the Hálslón Reservoir to the full supply level (FSL) at an elevation (EL) of 625 m. Water from the Hálslón Reservoir is conveyed through a 40-km long headrace tunnel to an underground powerhouse located in the Fljótsdalur Valley. The powerhouse has an installed capacity of 690 MW.

### *Dam Features*

The Kárahnjúkar Dam has a height of 198 m and a 700-m-long crest. A general plan view and elevation are shown in Fig. 5. The material zoning is presented on a cross section in Fig. 6. Given the previous events on cracking of high CFRDs during reservoir impounding, concerns arose over the potential for the Kárahnjúkar Dam to experience similar face-slab distress and cracking.

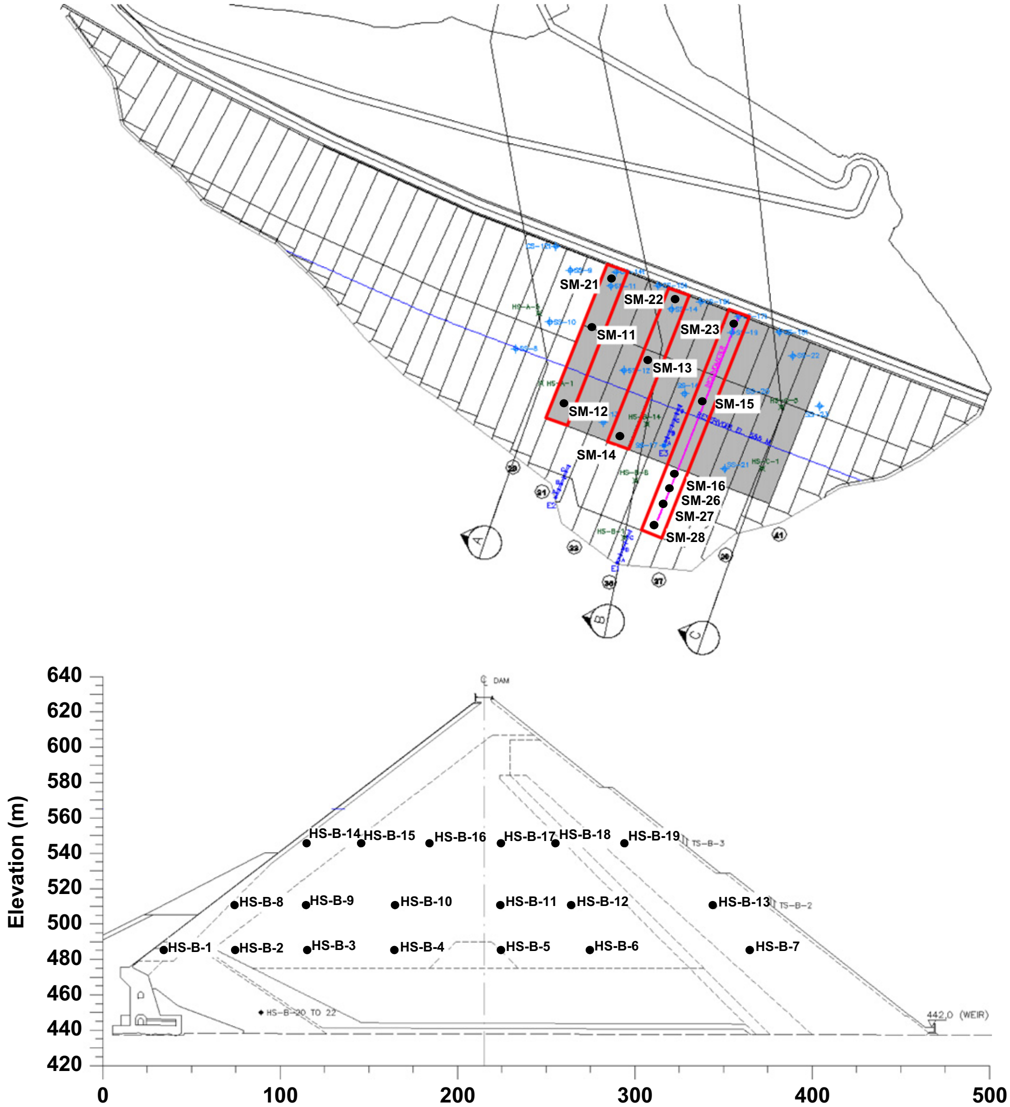



**Fig. 5.** Kárahnjúkar Dam plan view and elevation

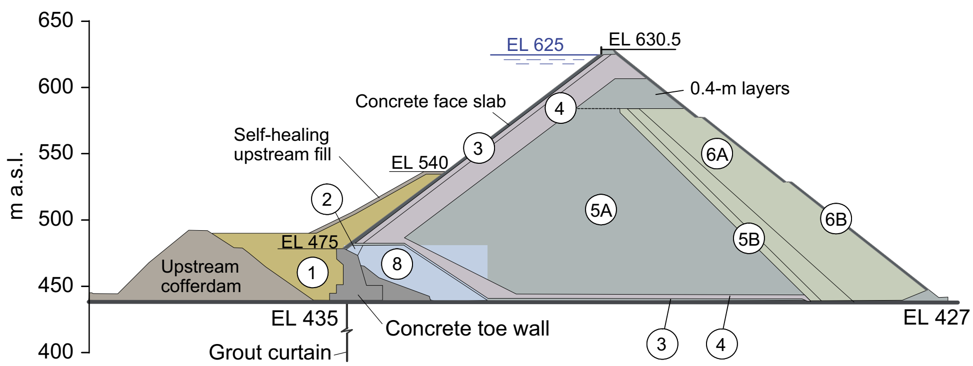



**Fig. 6.** Material zones within the rockfill

One particular feature is the deep canyon crossing the base of the dam. A concrete toe wall was provided at the base to facilitate construction and reduce the length of the slabs. This feature is rather atypical but convenient in this situation. Upstream of the dam, a self-healing fill was specified to protect the concrete slabs and also to fill any cracks if they were to develop.

### Schedule of Construction

The construction sequencing for rockfill placement and concrete face slabs of the CFRD was based on the construction scheduling and planning. The placement of the rockfill by dates is presented in Fig. 7 and the construction sequence for the slabs in Fig. 8.



Int. J. Geomech., 2016, 16(1): 04015024

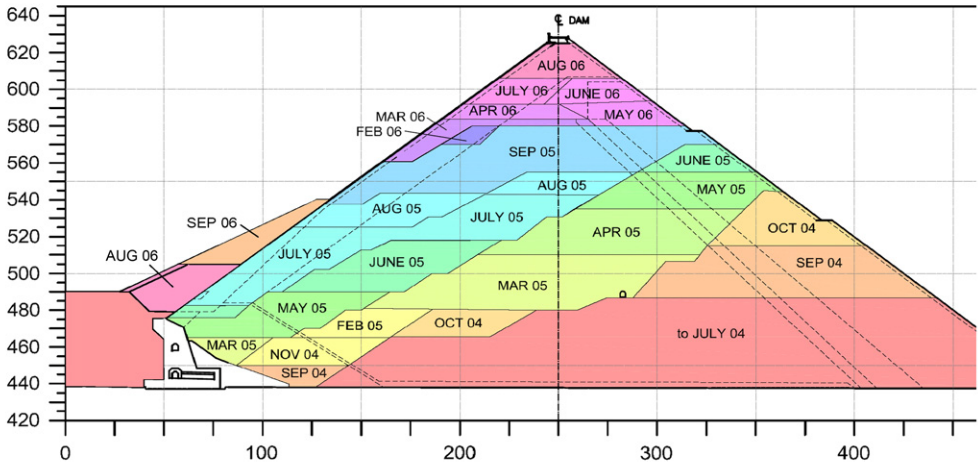



**Fig. 7.** Construction staging at maximum section: rockfill

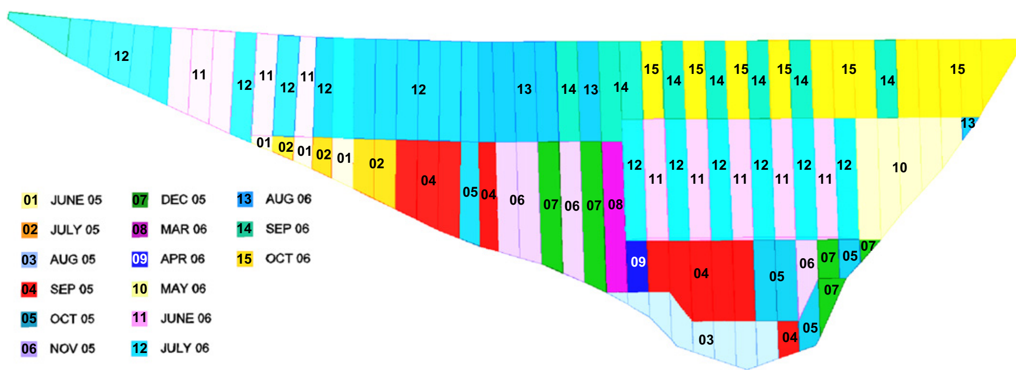
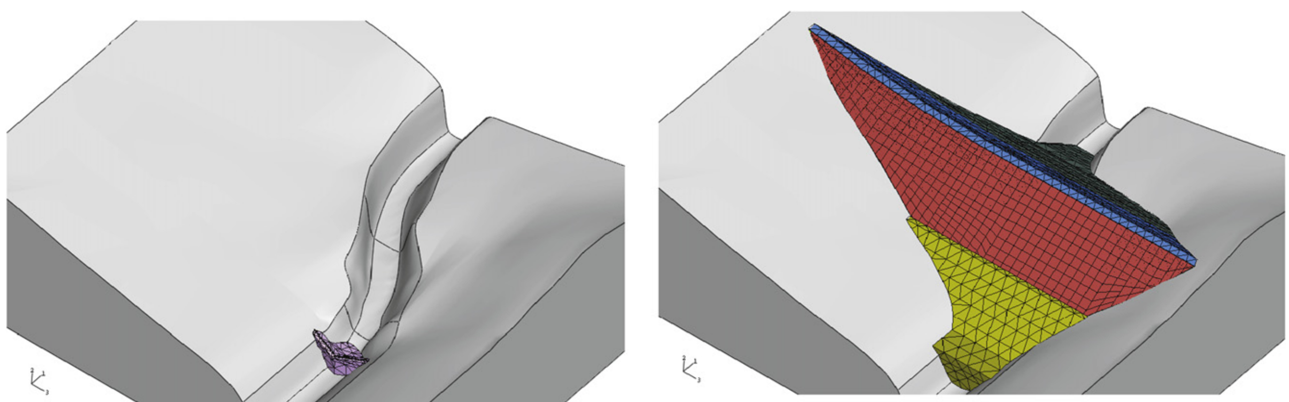



**Fig. 8.** Construction staging for concrete faces



(a) (b)

**Fig. 9.** (a) Three-dimensional view without dam; (b) 3D view showing the dam

## *Finite-Element Analysis of Kárahnjúkar CFRD*

This CFRD was analyzed with 3D solid elements using the FE software *Abaqus*. The 3D analysis was used for capturing the slight arched geometry and the pronounced canyon crossing the base. Figs. 9(a and b) schematically show the valley with and without the dam. The analysis determines the horizontal compression stress components identified as critical on other failed CFRDs. The slab thickness was chosen by previous experience, and adjustments were made to thicken the central portion of the facing.

Given the highly complex geometry, linear tetrahedron elements were used for meshing. The elements in the model were fitted to material zones and construction stages. Particular attention was given to the concrete facing and the supporting zones where the shear stresses are transferred between the rockfill and the concrete slabs loaded with hydrostatic pressure. Mesh coarseness was adjusted on the basis of zone behavior. For instance, the mesh for the downstream side of the dam is made coarser because the study is focused on cracking at the upstream facing.

This dam, similar to other CFRDs, was constructed in layers, and it is stiffer on the horizontal plane than the vertical plane. Furthermore, given the differences between the vertical and transverse moduli of elasticity, the material constitutive model chosen for the rockfill was the transverse isotropic stress–strain model that incorporates material anisotropy, and the concrete slabs were modeled as linear-elastic. The vertical modulus of

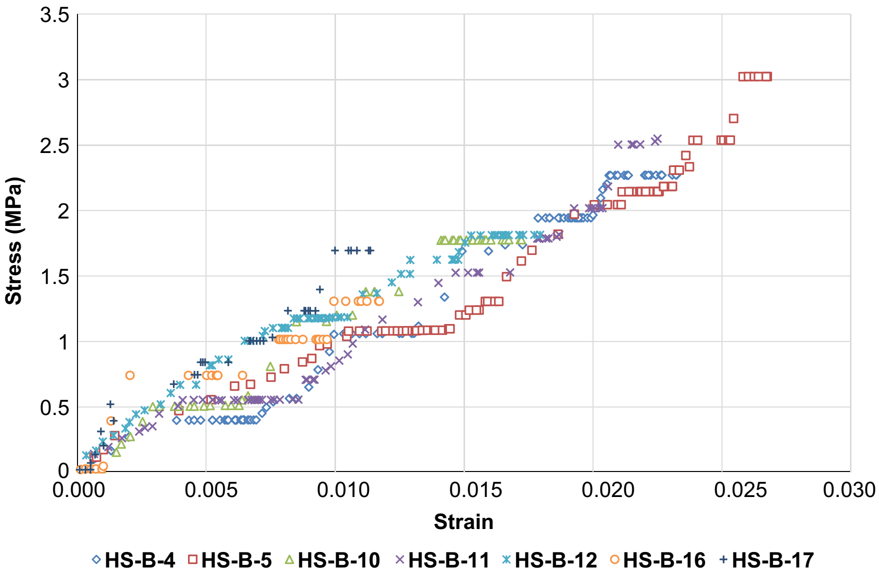



**Fig. 10.** Stress–strain relationship from measurements

**Table 2.** Material Properties

| Material | Density (kg/m$^3$) | Vertical $E$ (MPa) | Vertical $G$ (MPa) | Vertical $\nu$ | Horizontal $E$ (MPa) | Horizontal $G$ (MPa) | Horizontal $N$ |
|---|---|---|---|---|---|---|---|
| Gravel fill | 2,245 | 78 | 136 | 0.3 | 425 | 170 | 0.25 |
| Rockfill (upstream) | 2,143 | 56 | 97 | 0.3 | 305 | 122 | 0.25 |
| Rockfill (center) | 2,143 | 16 | 28 | 0.3 | 87 | 35 | 0.25 |
| Rockfill (downstream) | 2,143 | 13 | 22 | 0.3 | 70 | 28 | 0.25 |
| Upstream fill above EL 490 | 2,245 | 150 | 58 | 0.3 | 150 | 58 | 0.3 |
| Upstream fill below EL 490 | 2,245 | 250 | 96 | 0.3 | 250 | 96 | 0.3 |

Note: $E =$ Young's modulus; $G =$ shear modulus; $v =$ Poisson's ratio.

deformation is estimated using settlement measurements. These settlements are measured by the hydraulic settlement gauges installed. For this purpose, the vertical stresses are calculated with the fill height ($D$) above the settlement cells and the strains as the settlement ($s$) over the heights of rockfill below ($H$). The measured stress–strain behavior of the CFRD rockfill appears to be roughly linear. The computed vertical modulus also exhibits a constant value as the normal stress increases. These measurements are presented in Fig. 10.

A complete geotechnical study was conducted, and the Mohr–Coulomb parameters were established; the results indicated very small plasticization at the plinth zone of the dam. Shear dilation and yielding was deemed minimal for this dam configuration. For this reason, the Mohr–Coulomb plasticity was not included for the final evaluation. Table 2 presents the calibrated material properties for the model. Because the foundation for this dam is much stiffer than the rockfill, the modeling did not include the foundation, and therefore the boundary conditions at the dam base were assumed to be fixed. The transferring mechanism from the rockfill to the concrete facing was done through the rockfill-facing interface. The contact friction was modeled among the concrete slabs, and between the slabs and the rockfill and upstream fill using the classical Coulomb friction formulation, where the friction resistance developed during the slippage of two surfaces is proportional to the normal pressure (hydrostatic load) on the contact surface times a friction coefficient.

These types of interfaces were also incorporated between the toe wall and the rockfill, where additional settlement is expected due the vertical configuration of the canyon. Furthermore, these contact interfaces allow the surfaces to open resulting in no tensile stresses at the interface.

**Table 3.** Maximum Settlement for CFRD

| Comparison | Maximum settlement (m) |
|---|---|
| Measured settlement recorded from instrumentation | 1.53 |
| Computed settlement from FEA | 1.30 |

The nonlinear analysis performed involved 100 analysis steps and approximately 28,000 elements. Given that the mesh is constantly changing during the construction period, the stiffness matrix was gradually updated step by step as the activation and/or deactivation of elements occurred.

Initially, the analysis was performed without slabs, and most elements were deactivated and gradually reactivated (without strains) at every stage of the analysis following the construction sequence. Concrete-slab elements were also activated at their respective time frame and coordinated with interface activations between slabs. This process continued until the EOC to obtain vertical settlements, which were calibrated as described in the next section.

### *Calibration*

To calibrate the model, the measurements from the instrumentation platform were used to determine more accurate material



Int. J. Geomech., 2016, 16(1): 04015024

properties. The initial vertical moduli were estimated using the geotechnical investigations and correlations with similar dams and materials used. Subsequent modifications were required per material zoning. The initial calibration of the material properties was performed to correlate the measured settlements with the analysis results at the EOC, focusing particularly on the settlement gauges located at the maximum section (Fig. 6), where the higher dam section and canyon are located. This provided a starting point followed by 20 iterations to match the measurements.

### *Instrumentation*

The instrumentation installed on the dam consisted of hydraulic settlement gauges (HS), crest settlement points (CS), extensometers, survey points (SS), an inclinometer, and joint and strain meters (JM and SM). For calibration purposes, the hydraulic settlement gauge results from construction were used for estimating the rockfill properties, whereas the strain-meter results were used for estimating the stresses on the concrete slabs.

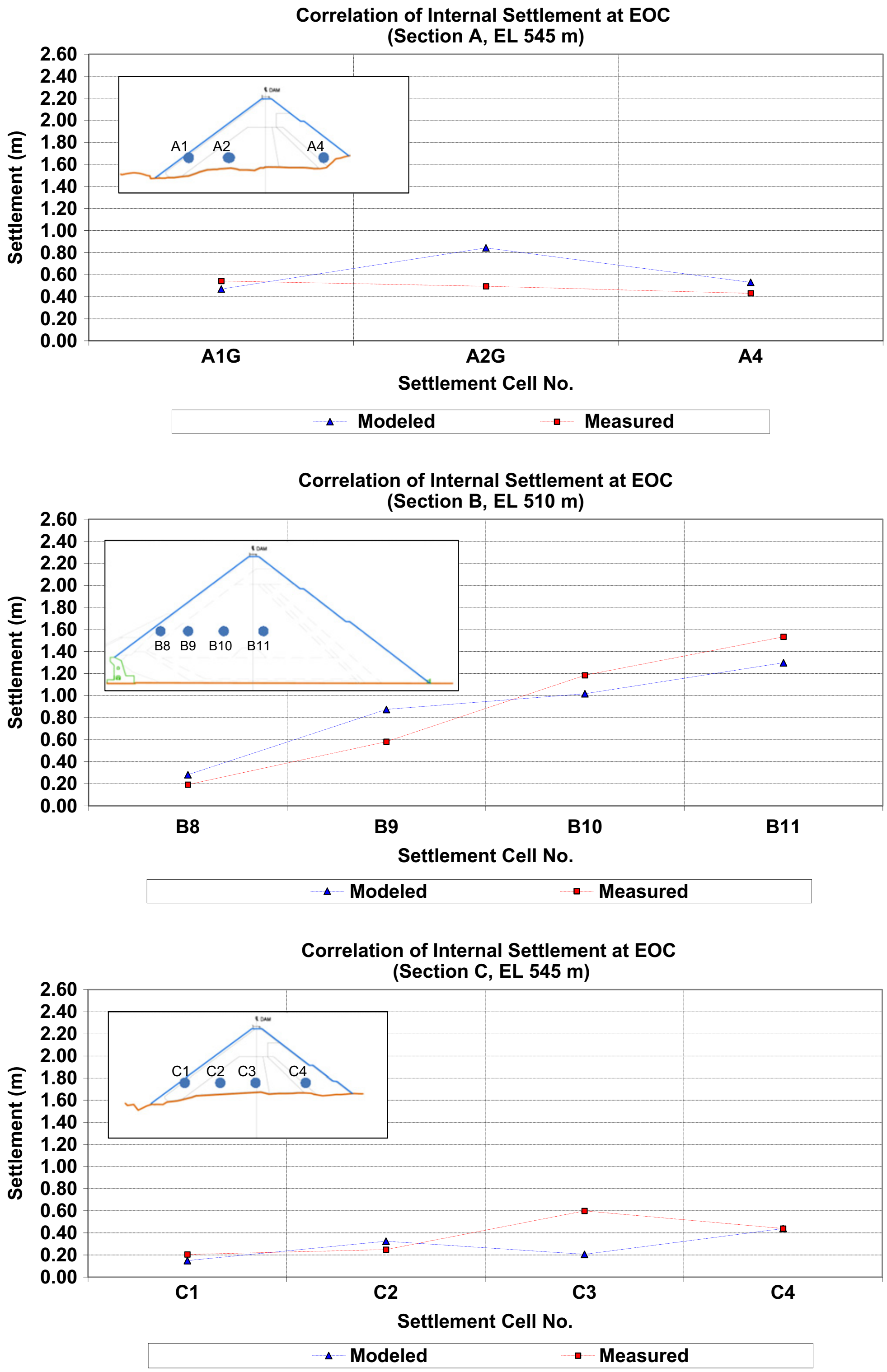



**Fig. 11.** Measured settlements versus calculated settlements

## Settlement Gauges

Hydraulic settlement gauges were installed for monitoring settlement of the embankment fill and face slabs. The settlement gauges measure vertical settlement of the fill below the installation elevation. The gauges were distributed along the three sections of the dam, shown in Fig. 6.

## Strain Meters

Strain meters were installed to monitor stresses and strains in the concrete face slabs of the CFRD. For the focus of this work, the strain meters located at the central portion aligned with the canyon are of greater interest, because the maximum compressive stresses are located on this area.

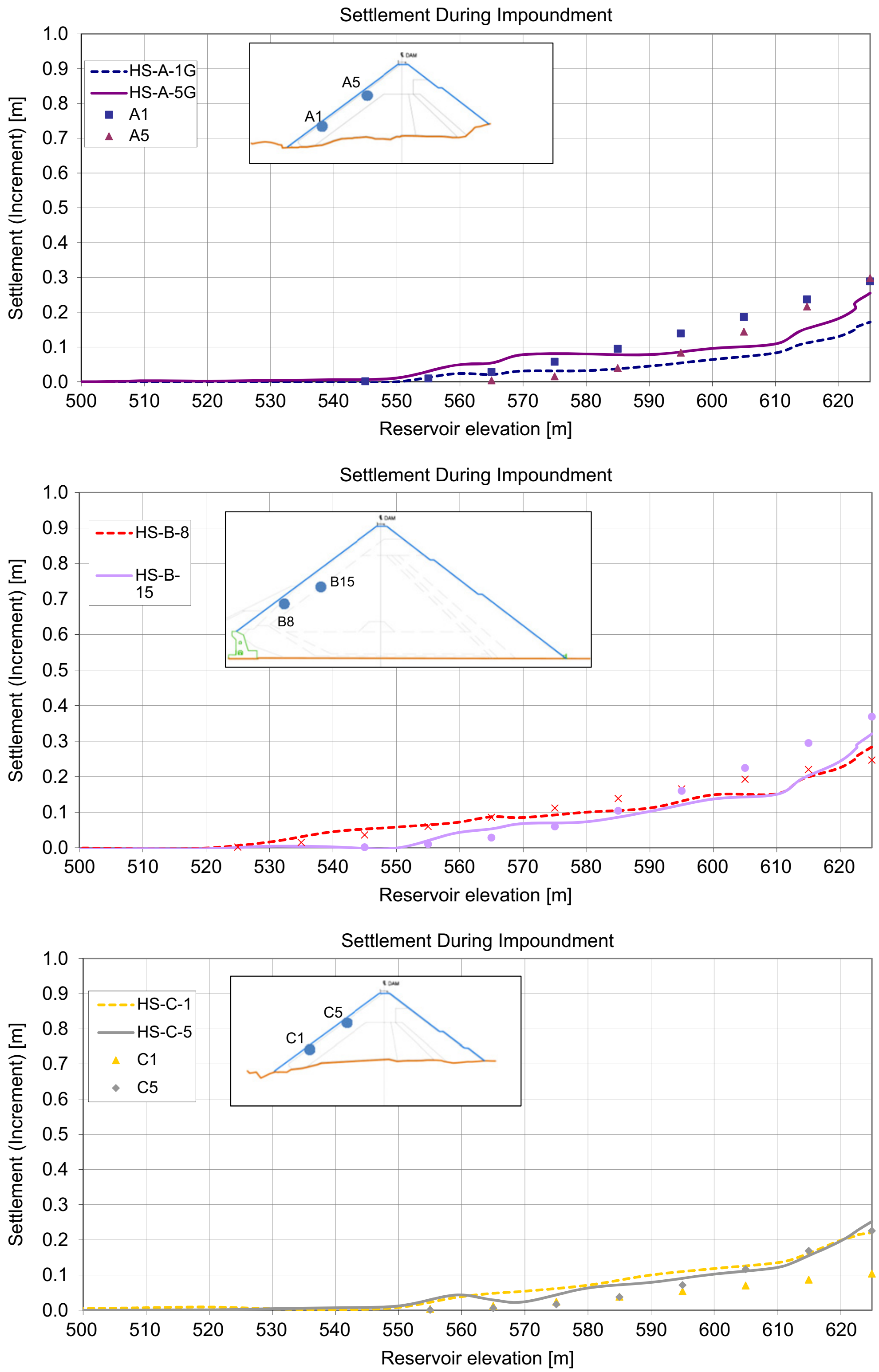



**Fig. 12.** Major settlements caused by impoundment at the three cross sections

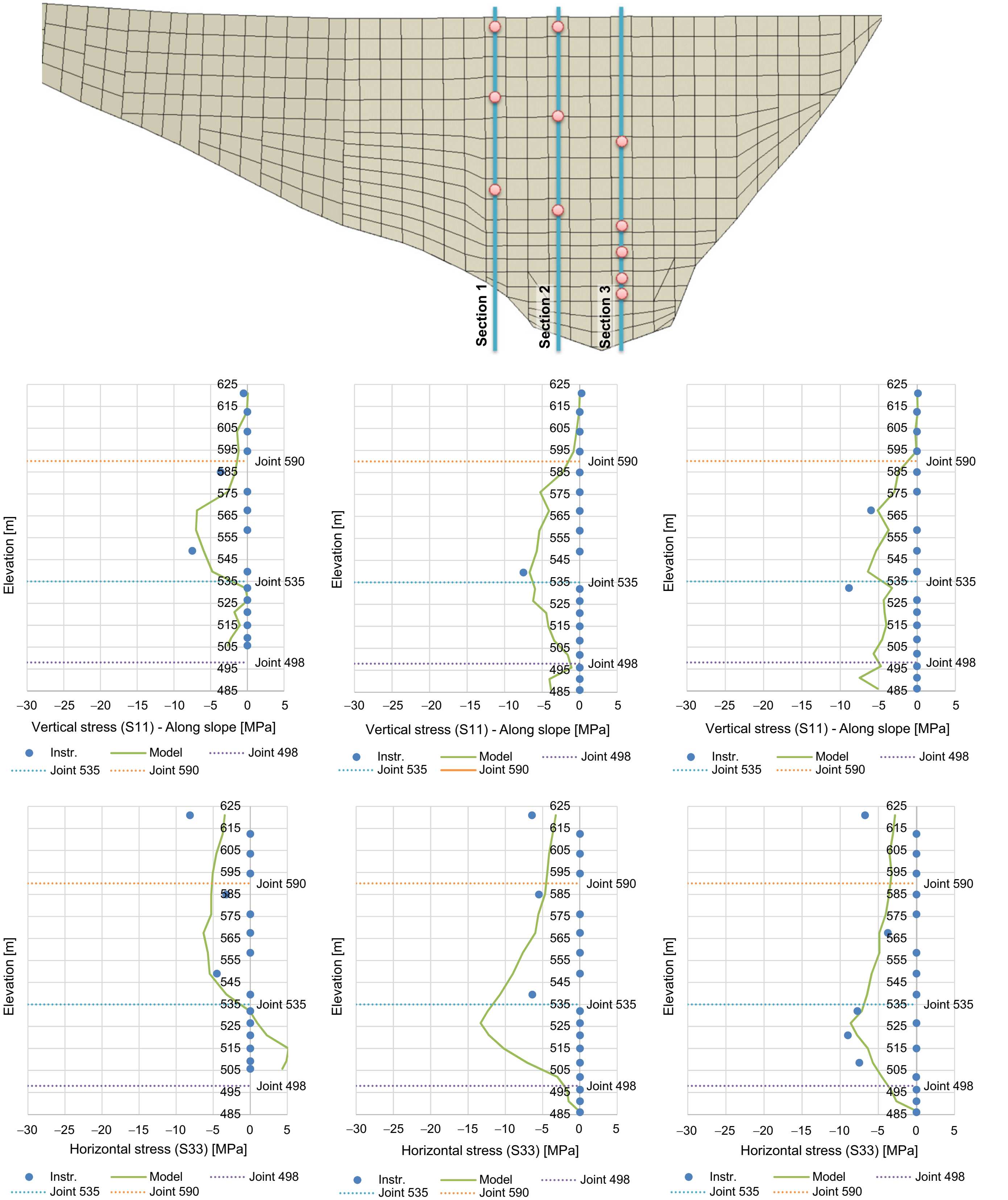



**Fig. 13.** Stresses caused by impoundment along three central slabs: measured versus computed

### *Results*

#### Settlements at the EOC

Table 3 presents the maximum recorded settlements at the EOC, used for calibration, for both measured and computed values.

Fig. 11 shows the relevant settlement comparisons at the EOC for Sections A, B, and C.

The results for the overall trend of settlement at the EOC show acceptable agreement between the instrumentation measurements and analysis results.

#### Settlements during Impounding

During the impounding phase, settlement values from instrumentation were correlated with analysis results. The values are presented as the difference of settlement between the current water level stage and the EOC until FSL was reached for the three sections. These settlement values reflect the change in settlement caused by the reservoir load. Fig. 12 shows the comparison of settlements during impoundment. Settlement values from instrumentation are depicted as dots, whereas analysis results are depicted as continuous lines.

The results for the settlement during impounding show acceptable correlation between the instrumentation measurements and analysis results.

#### Slab Strains Correlation

Measurements from strain meters installed adjacent to the central area of the dam, where the highest strains were recorded during impoundment, were used to validate the analysis results. Then, stresses on the slabs were quantified using strain measurements. The differential strains are compared (measured versus computed) for the three sections shown on along the central slabs (Fig. 13). The vertical red lines show the ultimate concrete strength.

The results for the strains in the slabs show acceptable agreement between the instrumentation measurements and analysis results. Also, the levels of induced stresses estimated from the analysis are significantly lower than the concrete ultimate strength.

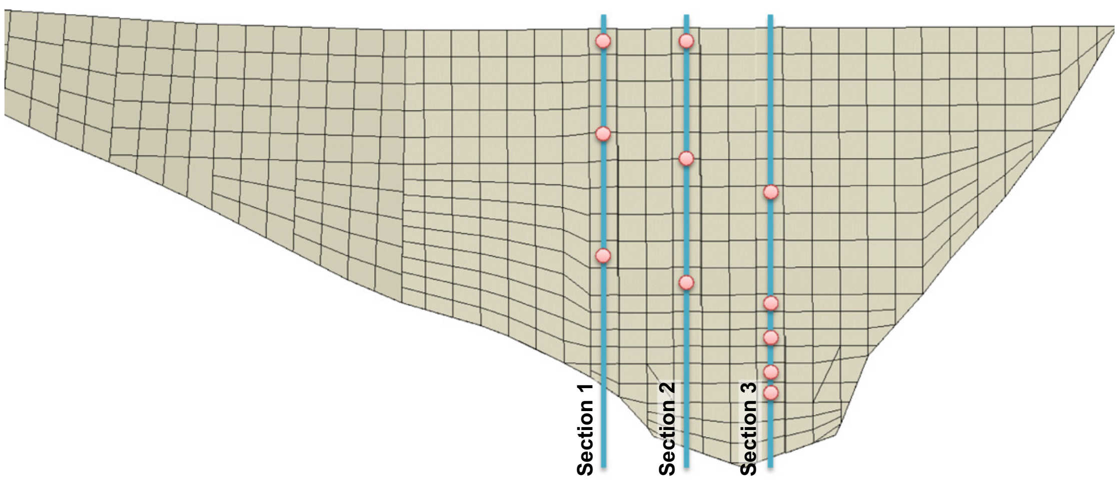



**Fig. 14.** Concrete slabs showing Sections 1, 2, and 3

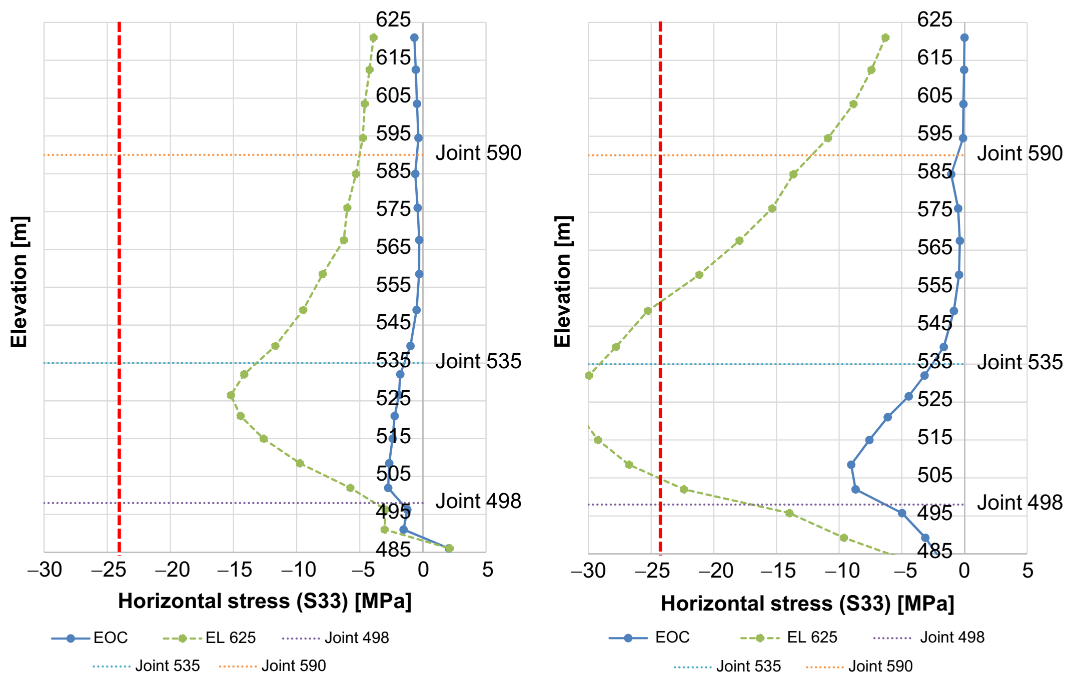



**Fig. 15.** Stress comparison through Section 2 with and without mitigation measures

This ascertains the integrity of the concrete face slabs of this CFRD. The low stress levels are the result of appropriate mitigation measures during construction suggested by the developed procedure explained below.

### *Mitigation Measures Suggestions*

To minimize the potential for cracks on the concrete slabs, some mitigation measures were taken into account for construction of the rockfill and face slabs for this dam: (1) reduction of lift thickness to stiffen the crest, (2) addition of a horizontal contraction joint, (3) consideration of a wider fiber spacer between vertical slab joints, (4) addition of an asphalt layer material to partially reduce the friction between slabs and rockfill, and (5) increase of central slab thicknesses by 10 cm at the central portion of the facing.

The benefits of these mitigation measures can be evaluated with the proposed analysis. Different scenarios can be compared in terms of their potential effect on the concrete slabs stresses. Failure in terms of slab cracking results when the stress demand reaches the concrete capacity. For instance, if no mitigations measures were taken, the resulting stresses would be much higher, with the likelihood of failure. For comparison purposes, the horizontal stresses from Section 2 in Fig. 14 are compared. If no mitigation measures were taken for this dam, the horizontal stresses from Section 2 would show values of around 31 MPa, which would exceed the compressive strength of the concrete used, 25 MPa. These results are presented in Fig. 15.

The overall distribution of stresses at the concrete facing is presented in the two main directions. Horizontal stresses (in the direction of the dam axis) are presented in Fig. 16, whereas slope stresses (in the inclined direction of the slope) are presented in Fig. 17. The horizontal stresses are higher at the lower third of the concrete facing and slightly shifted toward the left abutment because of the canyon geometry. Slope stresses are higher at the lowest portion of the slabs.

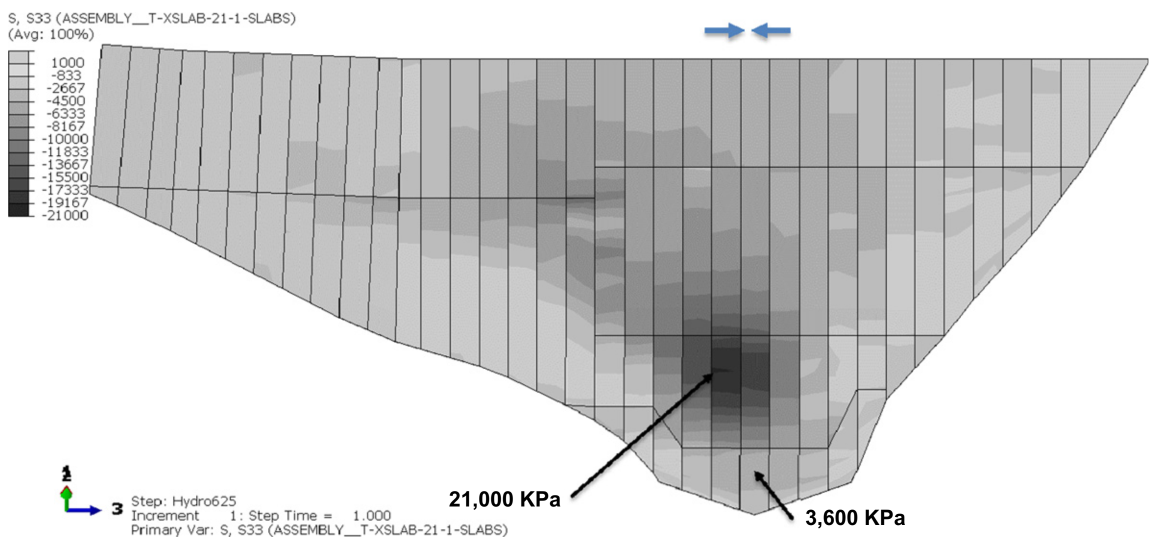



**Fig. 16.** Horizontal stresses on the slab at FSL

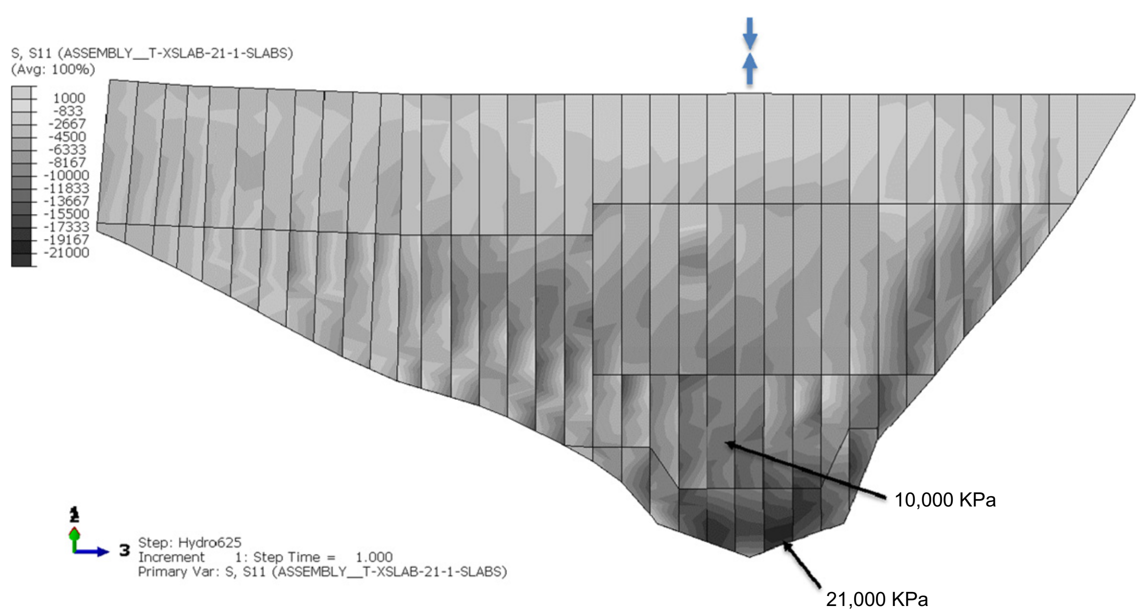



**Fig. 17.** Slope stresses on the slab at FSL



Int. J. Geomech., 2016, 16(1): 04015024

## Conclusion

In this work, a new FEA-based framework for analysis of new and during-construction CFRDs is developed. This stage-based analysis procedure has the flexibility to evaluate various alternatives on a new dam to achieve an optimum design and to incorporate changes that might occur during construction, leading to a more refined design that is more consistent with actual behavior. Because of the developed procedure's versatility, the capabilities of weighing different scenarios for cost–benefit evaluations are vast. The practicality and applicability of this framework makes it attractive for the design of new and during-construction CFRDs.

## References


*Abaqus*. [Computer software]. Providence, RI, Dassault Systèmes.

Anon. (2009). "Taking a tour on CFRDs." *Int. Water Power Dam Constr.*, 61(10), 22–30.

Cooke, J. B. (1984). "Progress in rockfill dams (18th Terzaghi lecture)." *J. Geotech. Engrg.*, 10.1061/(ASCE)0733-9410(1984)110:10(1381), 1383–1414.

Cooke, J. B., and Sherard, J. L. (1987). "Concrete face rockfill dam: II design." *J. Geotech. Engrg.*, 10.1061/(ASCE)0733-9410(1987)113:10(1113), 1113–1133.

Dolezalova, M., and Hladik, I. (2011). "Constitutive models for simulation of field performance of dams." *Int. J. Geomech.*, 10.1061/(ASCE)GM.1943-5622.0000039, 477–489.

Fitzpatrick, M. D., Cole, B. A., Kinstler, F. L., and Knoop, B. P. (1985). "Concrete face rock-fill dams–Design, construction, and performance." *Proc., Symp. on CFRD-Design, Construction and Performance*, J. B. Cooke and J. L. Sherard, eds., ASCE, Reston, VA, 410–434.

Johannesson, P. (2006). Fast-track construction of the Kárahnjúkar CFRD in Iceland, Commission Internationale des Grands Barrages (ICOLD).

Johannesson, P. (2007). "Modified zoning and compaction requirements for future high CFRDs." *Int. Symp. on Key Techniques for Dam Construction*, China, 26.

Larese, A., Rossi, R., Oñate, E., Toledo, M., Morán, R., and Campos, H. (2013). "Numerical and experimental study of overtopping and failure of Rockfill dams." *Int. J. Geomech.*, 10.1061/(ASCE)GM.1943-5622.0000345, 04014060.

Ma, H., and Cao, K. (2007). "Key technical problems of extra-high concrete face rockfill dams." *Sci. Chi. Ser. E: Tech. Sci.*, 50(Suppl 1), 20–33.

Marquez, P. L., and Pinto, N. (2005). "CFRD dam characteristics learned from experience." *Int. J. Hydropower Dams*, 1(1), 72–76.

Pinto, N. (2009). "The design and construction of extra high CFRDs." *Int. J. Hydropower Dams*, 16(3), 41–44.

Pinto, N., and Marquez, P. L. (1998). "Estimating the maximum face deflection in CFRDs." *Int. J. Hydropower Dams*, 5(6), 28–30.

Schreppers, G., and Lilliu, G. (2009). "DIANA analysis of a CFRD." *10th Benchmark Workshop on Numerical Analysis of Dams.*, ICOLD.