Mitigation Measures Evaluation for Concrete Faced Rockfill Dams

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Abstract: One of the most common types of dams is Concrete Faced Rockfill Dams (CFRD's). With higher CFRD's, some dams have experienced considerable fractures at the concrete faces, where in some instances these cracks have led to dewatering of the reservoir 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 CFRD's 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 paper, a new approach for analysis of CFRD's is presented. A comprehensive non-linear finite element analysis (FEA) 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. A case study using the developed framework is analyzed, the results are validated by the field measurements, and mitigation measures suggestions are provided. This methodology, based on the results of the investigation, provides guidelines and establishes a framework for analysis of CFRD's that can be used for design purposes and prevent any cracking of the concrete faces.

Keywords: Dam, CFRD, Finite Element Analysis, Mitigation

1. Introduction

Concrete Faced Rockfill Dams (CFRD's) are one type of embankment dams that are 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 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 Figure 1 (Cooke and Sherard 1987).

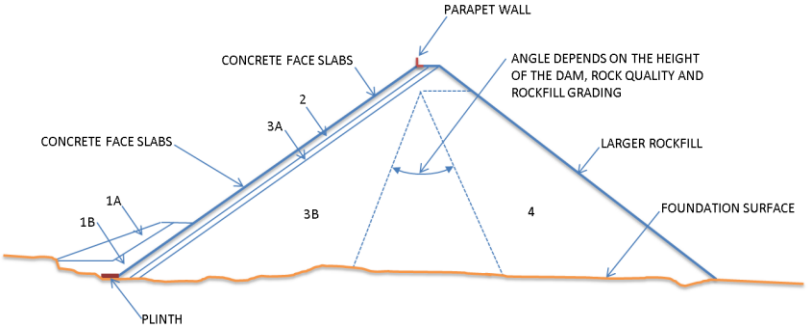

Figure 1. Typical cross section of a CFRD.

Zones 1A and 1B protect the upstream concrete faces. Zones 2A and 2B support the concrete faces. 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 most during construction. Additional zoning is defined as required.

Compared with other types of dams, CFRD's are straightforward to construct, economical, generally adaptable to terrain geometry; and materials are usually available in close proximity. However, some high CFRD's, such as Campos Novos (Brazil, 202m height), Barra Grande (Brazil, 185m height), Mohale (South Africa, 145m height), Aguamilpa (Mexico, 187m height), and Tianshengqiao (178m, China) have experienced significant structural failures as concrete slab fractures causing considerable leakage (Ma and Cao 2007).

The rockfill's flexibility compared with the stiffness of the concrete membrane directly impacts 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.

Despite the popularity of CFRD's, designs are mostly based on common practice rather than rigorous analysis procedures (Cooke 1984). However, due to the experienced structural failures of CFRD's, a more comprehensive methodology for analysis and design is needed. Furthermore, because of the 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.

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 198m, a length of about 730m and an installed capacity of 690MW (Johannesson 2006). The analysis includes the staged construction (resulting in an updated and larger stiffness matrix at every step of the analysis), 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, mitigation measures suggestions are provided. The method developed as well as mitigation measures can be followed in other similar cases and establishes an analysis framework for these types of dams.

2. Challenges on CFRD Design

2.1. Current Practice

Rockfill deformation 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, Ev, is obtained from vertical settlements (Fitzpatrick et al. 1985). Also, some empirical approaches allow the estimation of face slab deformations (Pinto and Marquez, 1998) by using the transverse modulus of deformation, Et. These two moduli were defined for the two phases: during construction and for first filling (Fitzpatrick et al, 1985). In many cases, the proposed empirical approaches relate the ratio Et/Ev with valley shape factor (A/h2), where A is the facing area, and h is the height of the dam.

Data from several constructed dams suggests that for narrower valleys, the measured settlements tend to decrease (Pinto and Marquez, 1998). This is observed when comparing the shape factor (A/h2) with the measured vertical modulus. This result indicates the presence of a stress arching effect across the abutments for narrow valleys and thus, emphasizing the importance of a three dimensional 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 towards or away from each other.

2.2. Problems Encountered

The main problem of CFRD's is cracking of the face slabs, which causes leakage leading to further damage to the rockfill body and loss of water. Estimation of rockfill settlements and face slabs deflections is essential on the analysis of a CFRD as 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. 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 tridimensional effect of the valley permits the rockfill movements towards the center of the dam that may induce additional tensile stresses of the rockfill and dragging the slabs at the abutments (Figure 2). 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 the concrete membrane deformations that may lead to crack development.

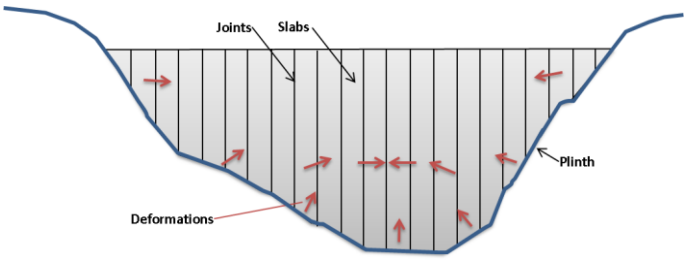

Figure 2. Face Slabs Dragged by the Rockfill Deformation.

2.3. Concrete Slab Cracking

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

Table I. Precedent CFRD's 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

Concrete slab cracking observed on several CFRD's presented 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 these compressive stresses and failures observed is related to the settlements during impoundment.

2.4. Methodology

The following diagram depicts a general procedure for analysis of CFRD's (Figure 3):

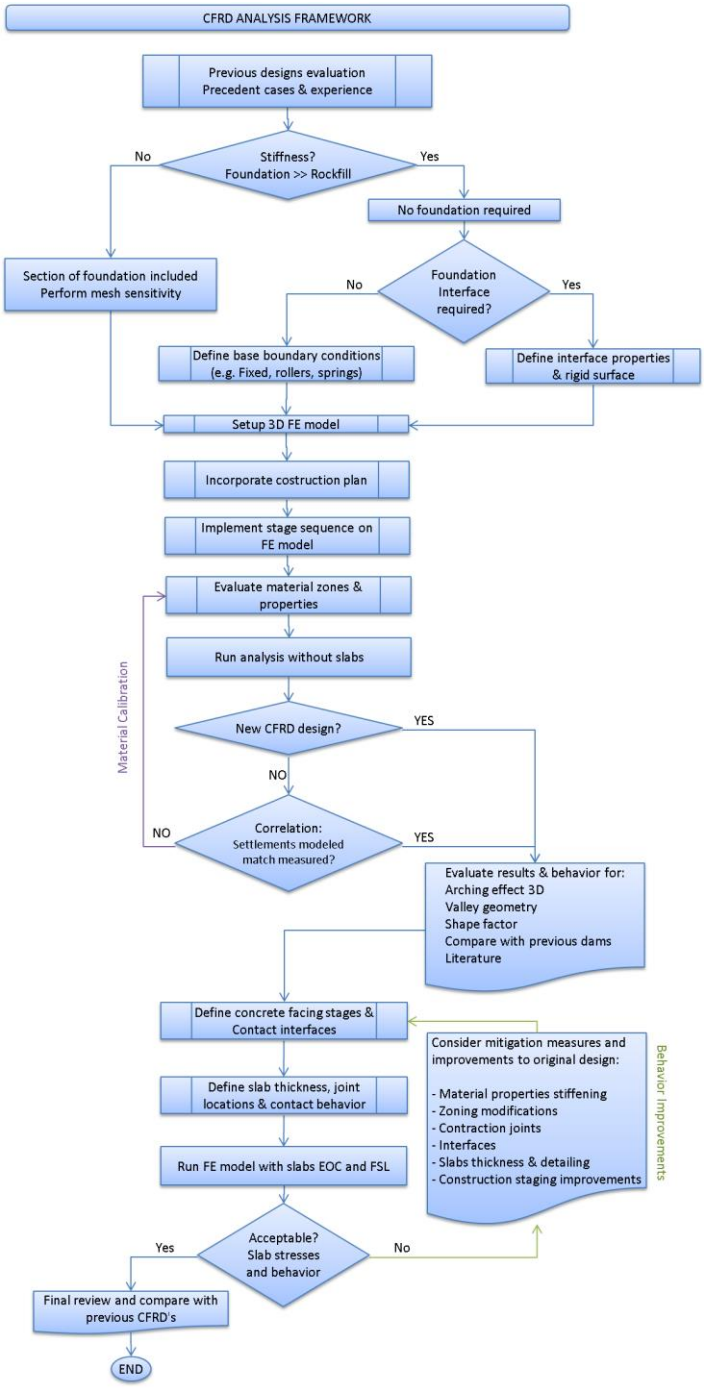

Figure 3. A general guideline for during-construction analysis of a CFRD.

If the dam is located on a narrow valley with a low shape factor, a three–dimensional (3D) model must be developed to properly capture the arching effect of the stresses distributed towards the abutments. The boundary conditions can be represented by a stiffness (spring), a rigid constraint, or a restrained boundary. 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 slide over the rigid foundation.

For the design of a new CFRD, construction staging is usually assumed based on 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 on the design phase due to 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 a realistic behavior.

The initial FE analysis 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. This process is repeated until a reasonable match with the measurements is achieved.

Once the model is calibrated, the concrete facing is included. The analysis results may suggest changes to the slabs thicknesses depending on the level of stresses. 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. The contact behavior of the joints depends on the filler material used, if any. The filler material can be made out of soft wood, EPDM fillers or other alternatives (Pinto, 2009). Later, the analysis results may suggest changes to the joints for an improved behavior such as an increased spacing and material behavior selection.

Once the model is ready, improvements on its behavior can be evaluated by an iterative process. First, settlements, slab deformations, and stresses need to be within an acceptable range in order 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
- Modify rockfill placement
- 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 a bond braker materials
- Increase slab thickness at selected locations

3. Case Study

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. At the dam site, the river is deeply incised in a canyon that is approximately 50m to 70m wide and about 50m 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.) 625m.

3.1. FEATURES

The Kárahnjúkar dam has a height of 198m and a 700m long crest. A general plan view is shown on Figure 4. The material zoning is presented on the maximum cross section through the canyon on Figure 5.

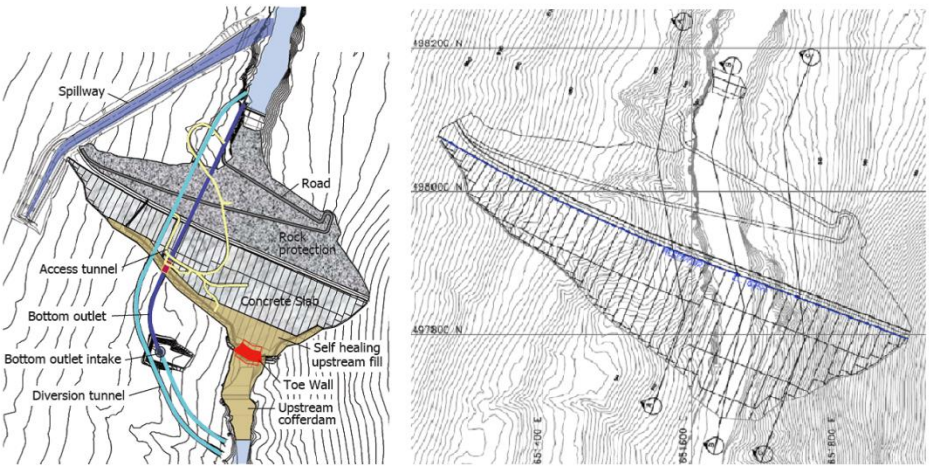

Figure 4. Plan view of the dam.

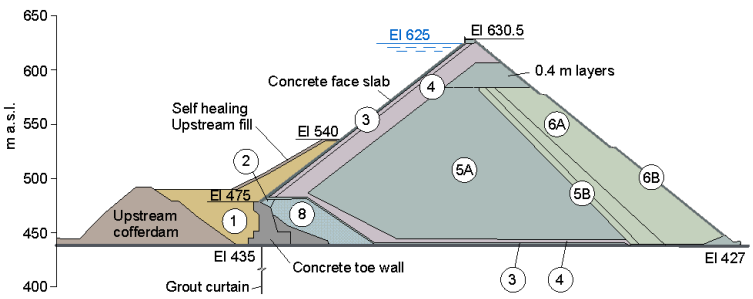

Figure 5. Kárahnjúkar material zoning.

3.2. 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 on Figure 6 and the construction sequence for the slabs on Figure 7.

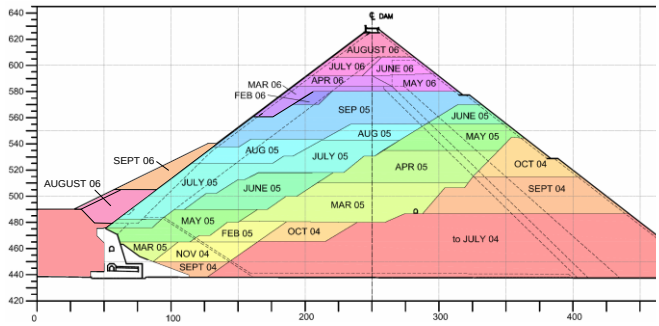

Figure 6. Construction Sequence for Rockfill.

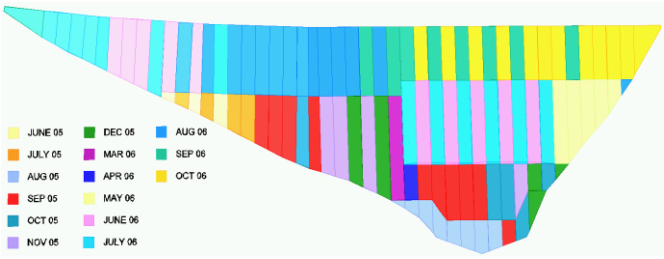

Figure 7. Concrete Facing Schedule.

3.3. FEA OF KÁRAHNJÚKAR CFRD

This CFRD was analyzed with 3D solid elements using the FE software ABAQUS. The 3D analysis captures the slight arched geometry and the pronounced canyon crossing the base. Figure 8 schematically shows the valley with and without the dam. The analysis determines the horizontal compression stress components identified as critical on other failed CFRD's.

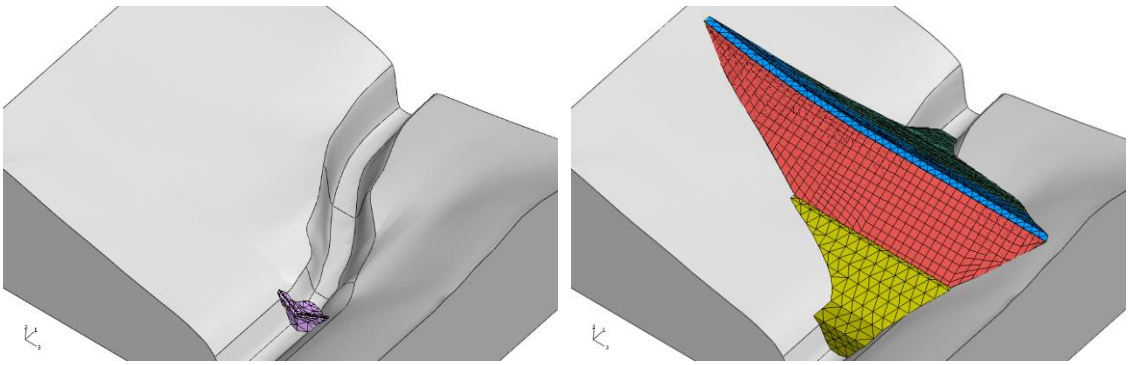

Figure 8. Valley with and without dam.

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. This dam, similar to other CFRD's, was constructed in layers and it is stiffer on the horizontal plane than the vertical. 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 deformation is estimated based on settlement measurements. 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.

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 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.

Table II. Material Properties

Material	Density (kg/m3)	Vertical			Horizontal		
		E (MPa)	G (MPa)	ν	E (MPa)	G (MPa)	N
Gravel Fill	2245	78	136	0.3	425	170	0.25
Rockfill (Upstream)	2143	56	97	0.3	305	122	0.25
Rockfill (Center)	2143	16	28	0.3	87	35	0.25
Rockfill (Downstream)	2143	13	22	0.3	70	28	0.25
Upstream Fill Above EL490	2245	150	58	0.3	150	58	0.3
Upstream Fill Below EL490	2245	250	96	0.3	250	96	0.3

E = Young's modulus; G = shear modulus; ν = Poisson's ratio

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. The non-linear analysis performed involved 100 analysis steps and approximately 28000 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 interfaces activations between slabs. This process continued to End of Construction to obtain vertical settlements which were calibrated as described on the next section.

3.4. CALIBRATION

The initial vertical moduli were estimated based on 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 B, where the higher dam section and canyon are located.

3.5. INSTRUMENTATION

For calibration purposes, the hydraulic settlement gauges results from construction were used for estimating the rockfill properties and strain meter results for estimating the stresses on the concrete slabs.

Hydraulic settlement gauges were installed for monitoring settlement of the embankment fill and face slabs.

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, since the maximum compressive stresses are located on this section (Figure 9).

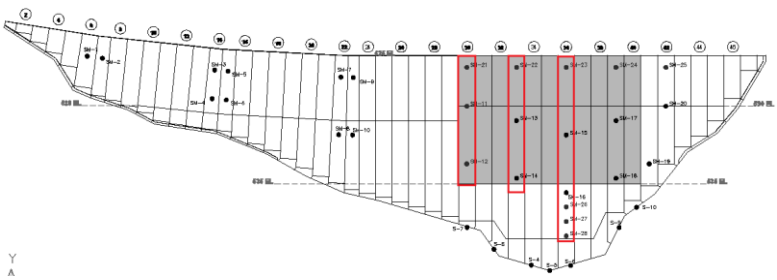

Figure 9. Concrete face slabs showing central strain meters.

4. Results

4.1. SETTLEMENTS AT EOC

Figure 10 shows the relevant settlement comparisons at EOC for all three sections A, B, and C.

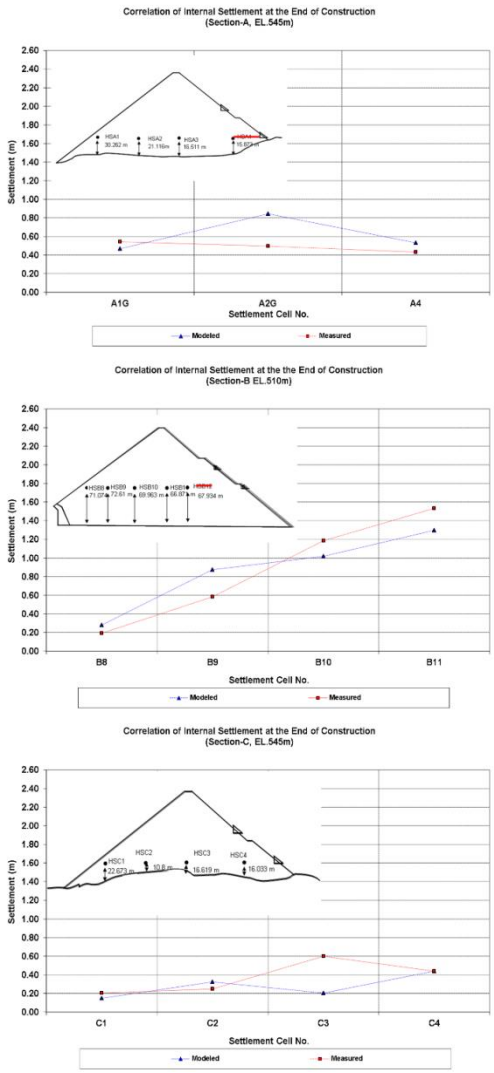

Figure 10. Major settlements due to impoundment at the three cross sections.

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

4.2. SETTLEMENTS DURING IMPOUNDING

During 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 EOC until FSL for the three sections. These settlement values reflect the change in settlement due the reservoir load. Figure 11 shows the comparison of settlements during impoundment. Settlement values from instrumentation are depicted as dots and analysis results are depicted as continuous lines.

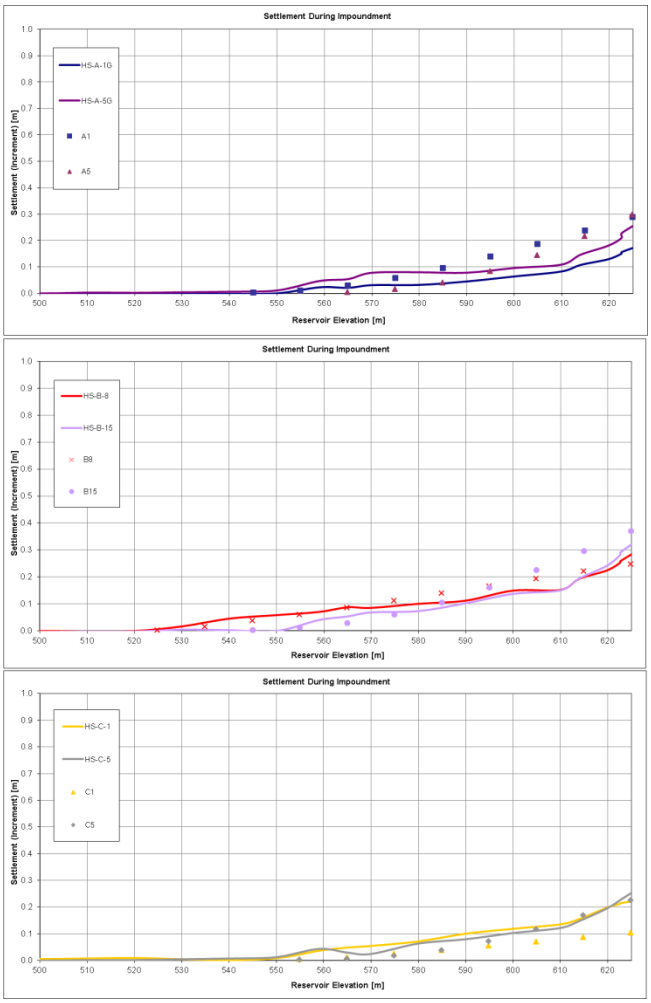

Figure 11. Comparison of settlements during impoundment.

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

4.3. 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 for validation of the analysis results. Then, stresses on the slabs were quantified based on strain measurements. The differential strains are compared (measured vs computed) for the slabs (Figure 12). The vertical red lines show the ultimate concrete strength.

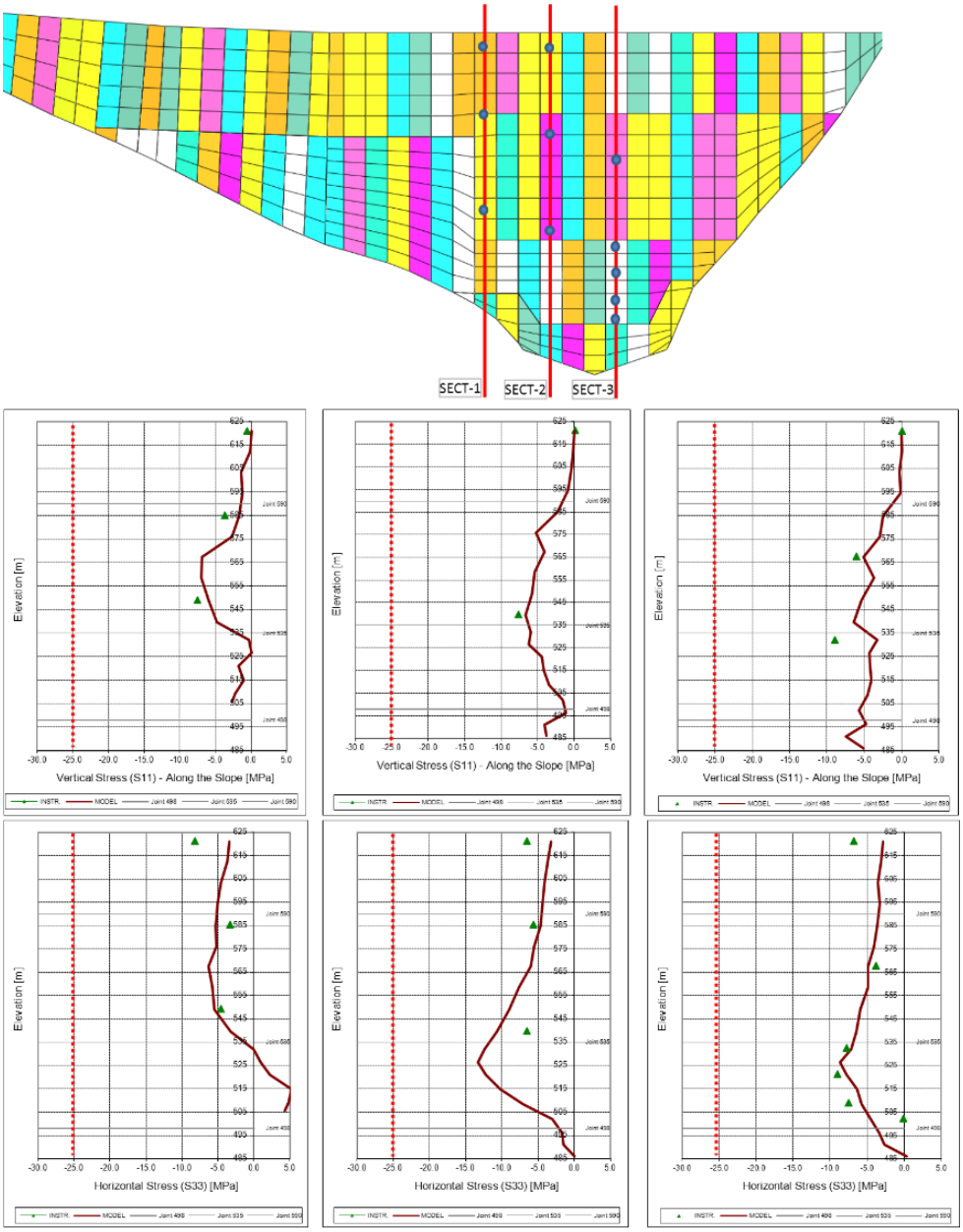

Figure 12. Impoundment stresses on central slabs – measured versus computed.

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

4.4. Mitigation Measures Suggestions

In order 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 in order 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 10cm 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 stresses demand reach the concrete capacity. For instance, if no mitigations measures were taken, the resulting stresses would be much higher, with the likelihood of failure. If no mitigation measures were taken for this dam, the horizontal stresses through section 2 would show values around 31 MPa which would exceed the compressive strength of the concrete used, 25 MPa. These results are presented on Figure 13.

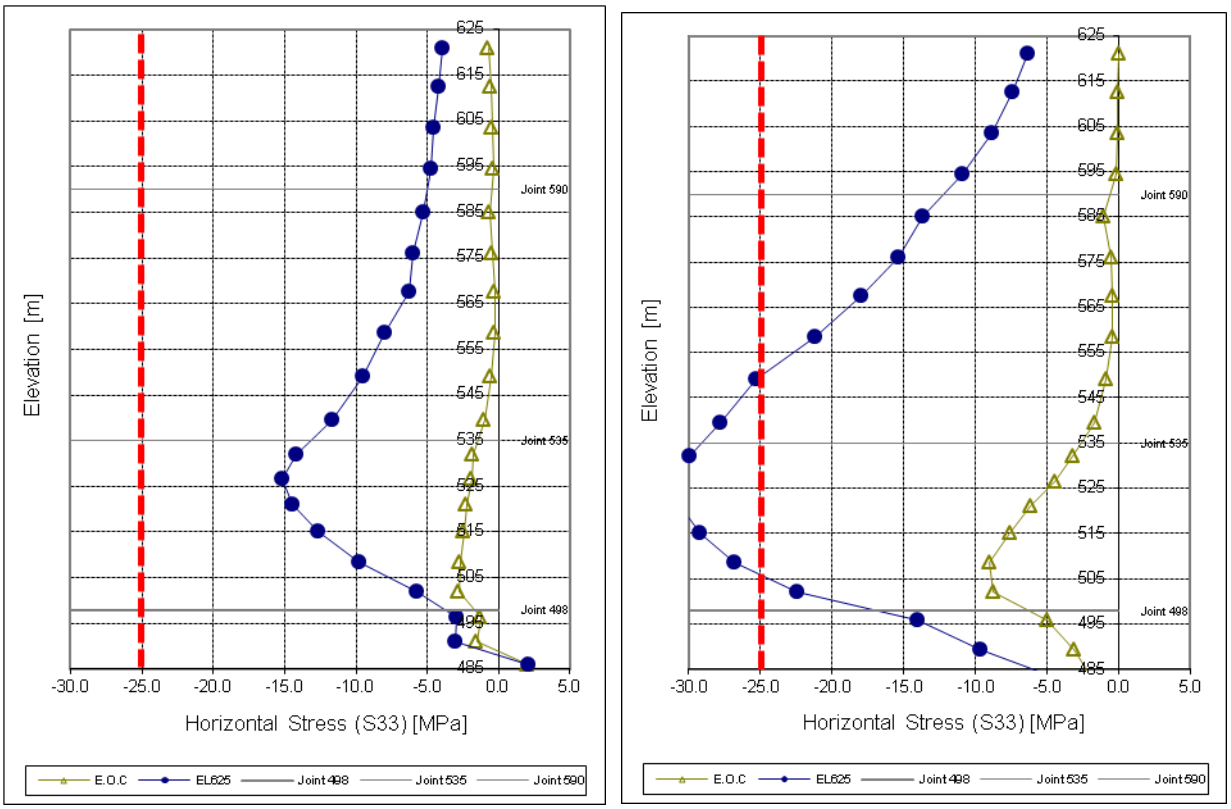

Figure 13. Comparison – Final Solution (left) vs No Mitigations (right).

5. Conclusion

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

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