DEFORMATION PREDICTION OF A LARGE CFSGD FOR FIRST IMPOUNDMENT

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ABSTRACT

Arkun Concrete Face Sand-Gravel Fill Dam is one of largest sand-gravel fill dams of the world. The dam is founded on a deep alluvium layer sealed by a deep cut-off wall. The impounding process started in December 2013.

In consideration of the foundation conditions and in respect to its height of 140 m the prognosis of deformation for the first impoundment was considered to be an important aspect of the assessment of the dam safety. For this purpose a 2D stress-strain model was prepared which was calibrated and verified in consideration of the available deformation measurements until end of construction. Magnetic ring settlement gauges, hydraulic settlement cells and topographical survey points were evaluated in order to obtain representative engineering parameters, particularly elasticity moduli, of the governing materials such as sand-gravel fill, rockfill and natural alluvium.

Arkun CFSGD is located in a narrow valley resulting in a valley shape factor of less than four which indicates that 3D stress redistribution effects might occur in form of arching. Hence, the obtained results were compared to data of benchmark dams worldwide in order to validate the prognosis. The deformation results of the 2D stress-strain model indicated conservative values compared to those benchmark data.

Concluding, the 2D numerical model was capable to describe the history of the monitoring devices quite well. The predicted deformations reflect a conservative assumption for the deformation predicting of first impoundment since 3D arching effects are not included.

Keywords: CFSGD, CFRD, sand-gravel, deformation, settlement, first impoundment

1. INTRODUCTION

A CFRD is one of the most popular dam types in the world. The free drainage character of rockfill material and the surface sealing are favorable in terms of the overall stability, seepage control and durability. Usually, the design engineer tries to find the optimum technical specification of the rockfill and sand-gravel fill materials in order to result in acceptable deformations and controlled seepage conditions. In this context the cracking and harms of the frequently applied surface concrete slabs are the critical aspects.

The technical specifications of the fill shall lead to an economic project and they shall guarantee that the face slab will not allow critical seepage flow and conditions within the dam body itself and the foundation. Since the face slabs are usually placed onto the surface after the fill works are more or less completed (End of Construction = EoC) the face slab is primarily loaded by the first impoundment. Corresponding to the impoundment strategy and the occurring inflow the face slab may relatively fast be subject to high water loads and, hence, high stresses/deformations. In order to control the stresses and potential harmful deformations the joint design and the number of joints in vertical and horizontal direction is also essential. Horizontal deformation joints are usually tried to be avoided.

The prediction of the behavior of face slabs during first impoundment is still quite difficult and several authors prepared studies which lead to results which are reasonable but are frequently done in form of a back-analysis after some problems occurred in regard to the performance of the face slab and occurring seepage. A simple but easy crosscheck of the expected deformations is the consideration of empirical data as given by Fell et al. (2005) or other authors.

Recently, 3D stress-strain analyses are performed which shall help to define the type of joints in terms of compression and extension and its specifications. Then the stresses in the face slab resulting from bending caused by relative deformation differences as a result of unfavorable geometry of the foundation and abutments shall be determined. Additionally, particular aspects and design issues can be investigated, e.g., the necessity of adjustment of the foundation and abutments to eliminate deformation differences and the necessity of additional joints, e.g., a horizontal joint.

2. DAM DESIGN AND ENGINEERING MATERIAL PROPERTIES

2.1. Design and zoning of the Arkun CFSGD

In Figure 1 a typical cross section is shown which corresponds to the latest design stage. The dam was founded partly on elevation 800.0 masl after excavation of up to 12.5 m of the alluvial layer. Further downstream of the dam's footprint the foundation was prepared by stripping a layer of 5 m. The plinth was placed onto an artificial fill reaching up to elevation 825.1 masl. Considering the occurrence of river bed rock at approximately 750 masl the cutoff wall reached a length of approximately 75 m.

Seepage and deformation control aspects dominated the design considerations of the Arkun CFSGD. Applying the existing river bed sand-gravel material is favorable in terms of deformations since sand-gravel fills show quite a favorable modulus of elasticity which is approximately 3-4 times higher than for good rockfills (Fell et al., 2005; Haselsteiner & Ersoy, 2011). Therefore the sand-gravel fill was placed within the upstream section of the dam close to the face slab in order to balance the deformations occurring during the first filling. For seepage control a sealing barrier consisting of three components was applied including a surface face slab, a deep cutoff wall and rock grouting. The connection of the surface slab and the cutoff wall was done by adding an articulated plinth.

The zoning of the dam is designed as follows. The upstream and center zones (3F, 3E) were formed by sand-gravel fill in order to control the deformations. An L-shaped drain (3H) was added in order to control the seepage already in the mid if the dam. This drain was protected by a suffusion filter (3H*) since the sand-gravel materials (3E, 3F) were considered to be slightly suffusive. The downstream slope was formed by coarse rockfill (3D) which was covered by riprap (RR). The rockfill (3G) also takes over a seepage control function which would still guarantee the slope stability if the sealing system would fail completely. The face slopes were selected typical for sand-gravel fill dams V:H = 1:1.6 on average. The downstream slope shows several berms which hosts the access road coming from downstream.

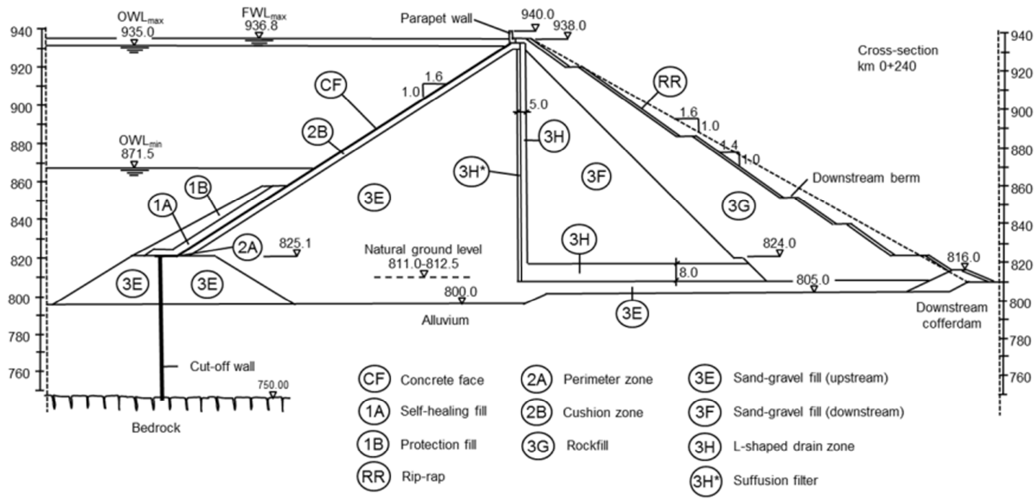

Figure 1. Typical cross section of the Arkun CFSGD

Different compared to ordinary CFRD designs Arkun CFSGD shows only 2A and 2B transition zones. Thanks to the application of the sand-gravel fill material the application of a 3A zone was not required. The downstream cofferdam was integrated into the main dam body. The upstream cofferdam is a rockfill dam with a clay core which was completed ahead of the main dam (see Figure 2 and Figure 3) and is located approx. 50 m upstream close to the river diversion tunnel inlet. According to the recommendations, e.g., given in ICOLD (2011), the plinth area was covered by an erodible, fine grained fill (1A) and a protection fill (1B) which was lead approx. to one third of the dam height.

2.2. Material parameters applied for the modelling

In Table 1 the applied material parameters are shown after they were calibrated by using actual monitoring data recorded during the filling period (see chapter 3). Crucial calibration parameters are marked in bold figures representing main fill zones which are 3E, 3F, 3G and the foundation layer (ALL). The refilled layer between the foundation level at 800 masl and the original ground level 812.5 masl was named F-ALL and its engineering properties needed to be defined differently to the adjacent materials 3F and ALL. Due to high groundwater level and quick construction progress at the beginning of the filling works the engineering parameters of F-ALL did not reach the favorable values of the fills 3E or 3F but were better than the natural alluvium (ALL).

For the main sand-gravel fill zones 3E and 3F the representative E-modulus was considered to be E = 220 MPa. For the rockfill zone 3G it is E = 80 MPa. For the natural alluvium in the Coruh River bed the calibration showed that an elasticity modulus of E = 150 MPa matches well in regard to the calibration and verification progress. The final E-modulus for the F-ALL zone reached E = 200 MPa after calibration was considered to be sufficient accurate.

Due to the coarse nature of the main fill parameters the friction angle used for the MC-model is quite strong and shows a range of φ' = 38-45°. It is noted that especially for high stresses the shear-strength behavior of fill materials is stress-dependent. Hence, a non-linear shear curve was used for modelling the shear behavior in the stability analysis. For the deformation modelling this behavior was neglected.

Due to the wide-graded sieve curve of the sand-gravel materials it showed very little pore volume and, therefore, quite high values for the unit weight γ = 24 kN/m³ compared to benchmark values of similar projects.

Table 1. Selected, calibrated material parameters

Models	Linear-elastic			Linear-elastic			Linear-elastic			Elastic-plastic (MC)				
Source	International consultant			Björnsen Consulting Engineers			Local design engineer			Björnsen Consulting Engineers				
Zone	E [MPa]	ν [-]	γ [kN/m³]	E [MPa]	ν [-]	γ [kN/m³]	E [MPa]	ν [-]	γ [kN/m³]	E [MPa]	ν [-]	γ [kN/m³]	φ' [°]	ψ [°]
1A	20	0,35	18	20	0,35	19	20	0,3	17	20	0,35	19	30	0
1B	30	0,3	18	80	0,33	21	200	0,3	21,5	80	0,33	21	35	4
2A	360	0,3	22,5	300	0,3	21	-	-	-	300	0,3	21	38	10
2B	160	0,3	22	250	0,3	21	200	0,3	22	250	0,3	21	40	10
3E	240	0,33	23	220	0,33	24	175	0,38	22	**220**	0,33	24	**38**	12
3F	200	0,33	22,5	220	0,33	24	200			**220**	0,33	24	**38**	12
3G	120	0,33	20,5	80	0,28	22	200			**80**	0,28	22	**45**	15
3H	120	0,33	19	100	0,3	19	150			100	0,3	19	38	4
3H*	-	-	-	100	0,3	19	-	-	-	100	0,3	19	38	4
ALL	200	0,33	22	150	0,33	19	660	-	-	**150**	0,33	19	**35**	8
ROCK	5000	0,15	26	20000	0,25	27	15000	0,2	26	20000	0,25	26	-	-
CON	20000	0,25	25	20000	0,2	25	28000	0,2	24	20000	0,20	25	-	-
F-ALL	-	-	-	200	0,33	24	-	-	-	**200**	0,33	24	**38**	10
RR	-	-	-	60	0,35	24	-	-	-	60	0,35	24	-	-
COW	-	-	-	800	0,27	23	-	-	-	800/1200/1800	0,26	24	-	-

The rock foundation (ROCK) is considered to be good since Andesite rock is present. The rockfill material (3G) is extracted from quarries located in the same bedrock type. In correspondence to rock-mechanical classification systems the present rock foundation is classified as "good" rock.

3. MODELLING AND CALIBRATION

3.1. General and constraints

The model prepared is a 2D stress-strain model applying two different soil behavior models, namely linear-elastic and elastic-plastic models. Those models are integrated in the Sigma/W software package of GEOSLOPE.

For the upstream and downstream model borders horizontal deformations were fixed whilst keeping the vertical flexible. The horizontal lower border was fixed in x- and y-direction. The borders up- and downstream were placed approx. two times the dam height away from the upstream and downstream dam toe, respectively. The same is valid for the modelled foundation thickness which was selected two times the dam height below the foundation elevation of the dam.

The cofferdam does not show any effect on the main dams overall deformations. However, it was modelled also in order to check the models functionality as double-check option.

The initial water level was set to 800 masl. Later when the cut-off wall emerged a sealing function the water table increased at least within the upstream area. This increase was neglected. Also fluctuating groundwater tables were not considered.

Figure 2 shows the zoning of the dam, elevations and the modelled area as well as the boundary settings.

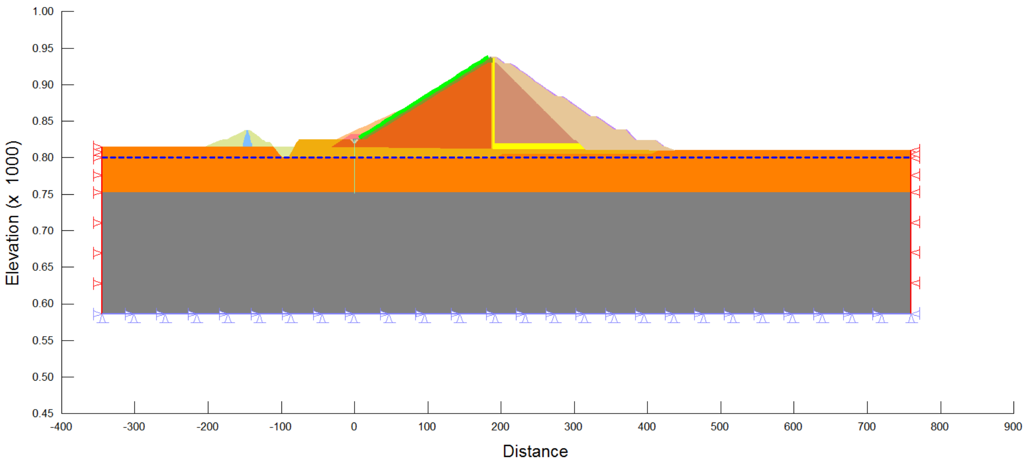

Figure 2. Dam model showing zones and boundary conditions (model phase 49)

The applied mesh is shown in Figure 3. The mesh size was 8 m on average. Refining of selected areas was performed in correspondence to the need of accuracy. Therefore, the areas close to the plinth, the cutoff wall, the face slab and close to the L-shaped drain were was refined.

3.2. Construction sequence, loading at first impoundment, cutoff wall

The construction sequence was modelled in layers of maximum 5-10 m step by step according to the actual construction progress records in 49 phases including the cofferdam and foundation excavation works. Phase 49 is End of Construction (EoC). Later, the first impoundment was modelled with one more step numbered 50.

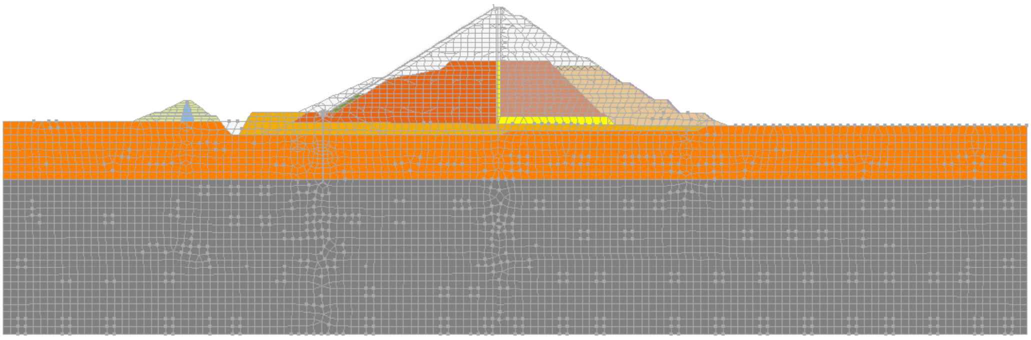

Figure 3. Applied mesh with an overall size of 8 m showing model phase 15 (fill status 04/2013)

First impoundment was modelled by the default water table load option as already implemented in Sigma/W and separately as node forces since the default feature can only apply loads onto the surface and not within the meshed area as visualized in Figure 4.

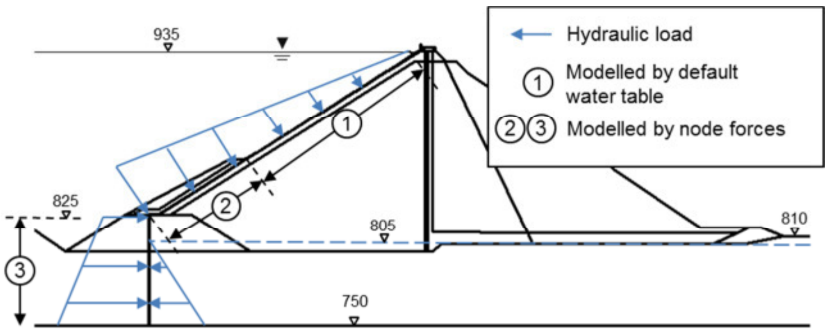

Figure 4. Load application for the first impoundment phase

As indicated in Table 1 the cutoff wall was modelled in correspondence to the construction sequence and its hardening development applying an initial E-modulus of E = 800 MPa and a final one of E = 1,800 MPa. The cutoff wall structures was activated when it reached almost a coherent body at that stage when it was assumed that it could take static loads before it was completed in August/September 2012.

4. DEFORMATIONS AT EOC AND CALIBRATION

4.1. Calibration

The calibration of the applied engineering parameters was done in consideration of the development of the monitoring data mainly considering the readings of the installed settlement cells (SC) and of the settlement gauges (SG). Additionally, the deformations of the surface points were used for cross-check. Two pieces of settlement gauges were installed later during the construction in order to complete the redundancy of the settlement monitoring system in the dam body. The calibration of the model was performed when the fill works hardly reached the elevation 900 masl in order to utilize the remaining deformation data for verification.

Due to late start of the read-out of the monitoring data – the fill started in May 2011 whereas the monitoring started in September 2011 approx. 3-4 months later – the records missed the initial deformation so that an adaptation of the data needed to be performed (Figure 5, Figure 6).

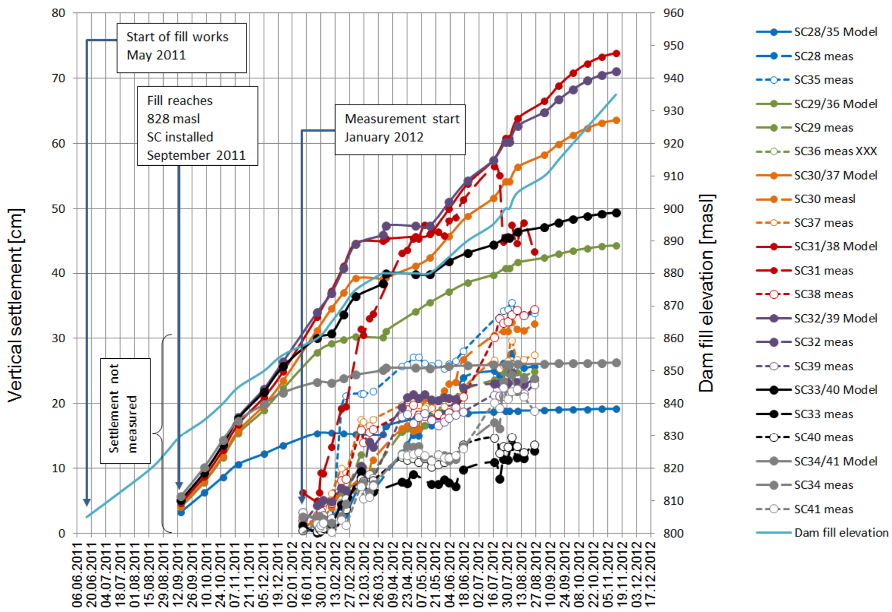

Figure 5. Load application for the first impoundment phase

After adding the initial settlement derived from the numerical dam model the development and absolute figures of the modelled and monitored data were compared and the mentioned calibration parameters (see Table 1) were adjusted as long as for all the SG and SC the derivation between the data sets were considered not to be acceptable. Generally, the model reached an accuracy of +/- 10 cm (Figure 6). Specific construction events such as the fill works stopped partially during the period February to May 2012 were modelled sufficiently (see also Figure 3).

During this work stop it was decided to refit the dam model with two magnetic ring settlement gauges (SG), as aforementioned, also in order to verify the monitored results of the hydraulic settlement cells. A comparison of selected data resulting of the model and the monitoring results are given in Figure 7. Although the monitoring data were suffering from slight inaccuracies resulting from interference of the survey works with the ongoing construction works the obtained accuracy was considered to be fit for purpose. Finally, the engineering properties presented in Table 1 were calibrated by both the SC and SG data and cross-checked by the surface survey point data.

4.1. Deformations at EoC

The deformations at EoC were already providing quite a reliable indication of the performance and the quality of the dam fill works. In Figure 8 and Figure 9 the vertical and horizontal deformations at EoC are shown. The vertical deformation reached approx. 0.75 m (Figure 8). These values are in line with the values of some case studies given in Hunter & Fell (2003). The displacement of 0.75 m at EoC corresponds to 0.5 % of the dam height which indicates an excellent behavior compared to the measured deformations of other dams such as Khao Laem or similar.

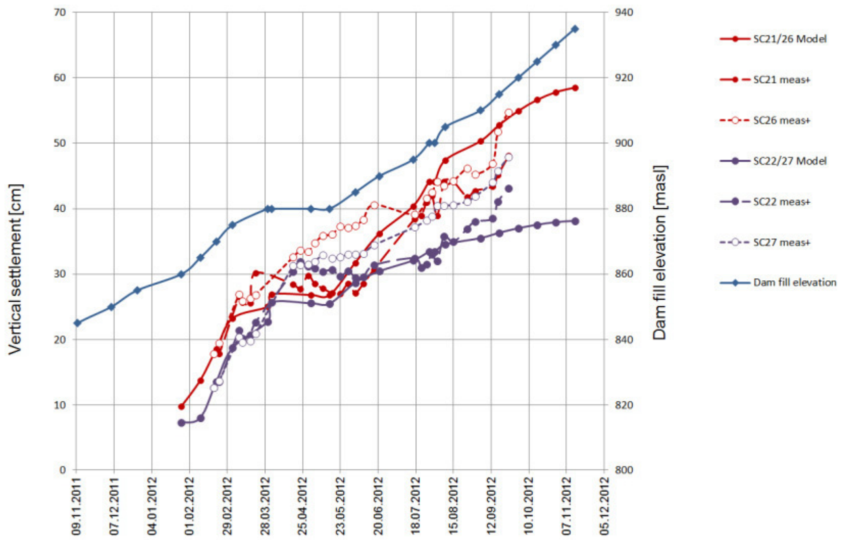

Figure 6. Modelled and measured settlements at SC 21/26 and SC22/27

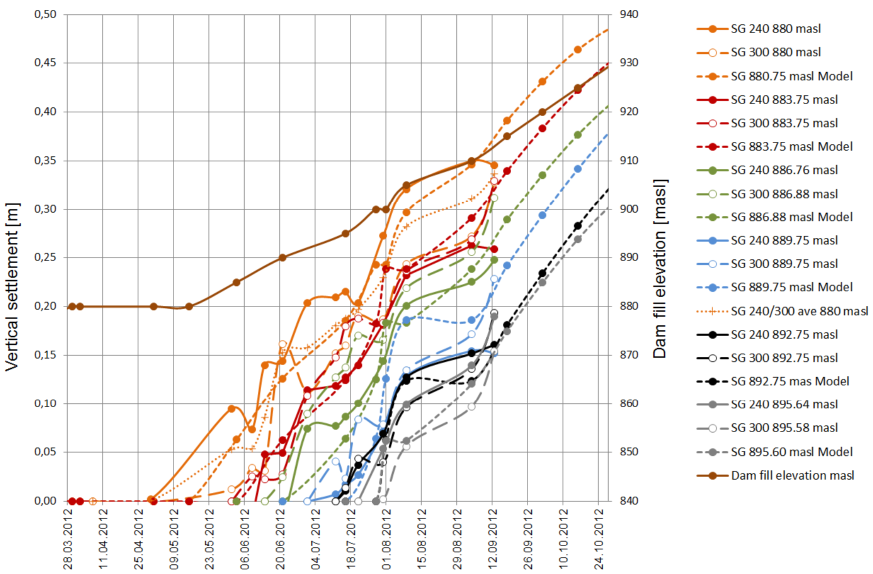

Figure 7. Deformations resulting from the magnetic ring settlement gauges and the corresponding modelling results

Maximum horizontal deformations reached a value of 0.20 m in the downstream shell and 0.15 m in the upstream shell (Figure 9). The rockfill shows a smaller elasticity modulus so that this is considered to be reasonable. At the surface the horizontal deformations were showing maximum values of 0.15 m at the downstream dam toe where also superficial survey points (SSP) were installed. The check comparison of the model and the monitoring data showed again excellent accuracy.

Both, the vertical and horizontal deformation pattern do reflect theory (Kutzner, 1996) but showing a transition of the maximum deformations at low elevations due to the presence of a soft foundation. Additionally, due to the specific construction sequence a specific stress and deformation behavior can be detected at elevation 880 masl.

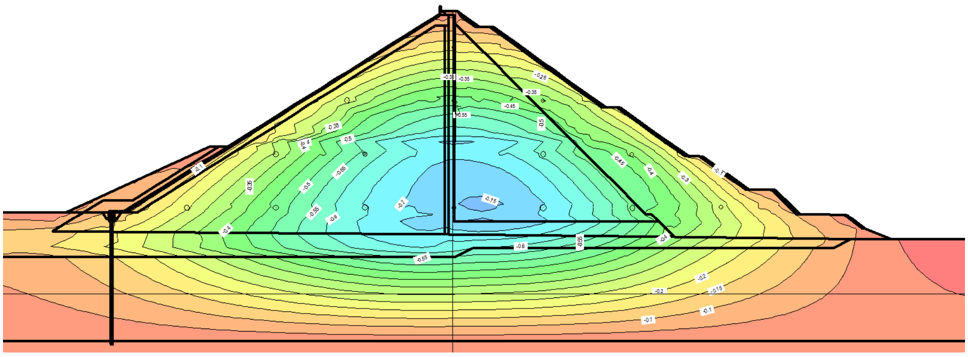

Figure 8. Vertical deformations [m] of Arkun CFSGD at EoC

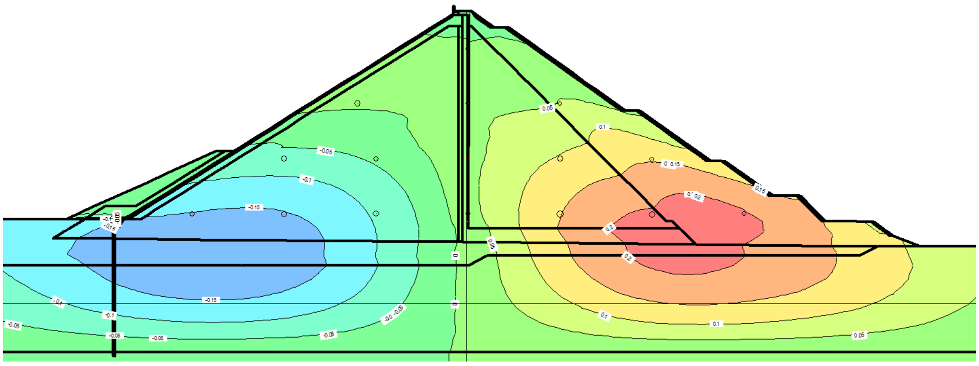

Figure 9. Horizontal deformations [m] of Arkun CFSGD at EoC

5. DEFORMATION AT FIRST IMPOUNDMENT

For the behavior of the face slab and other critical structures of the dam the EoC reflects the initial state for the first impoundment which is usually critical in terms of the occurrence of harmful deformations leading to cracks or similar disadvantageous processes. The total displacements reach a value of maximum 0.45 m at the plinth area where the face slab is connected to the articulated plinth (Figure 10).

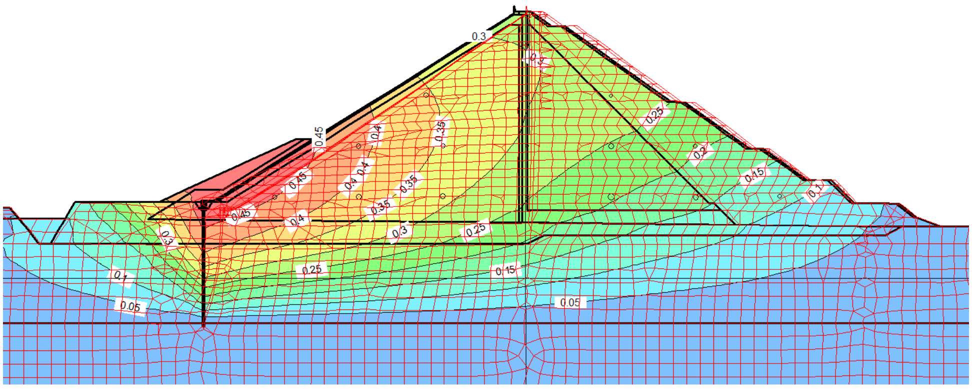

Figure 10. Total displacements and deformed mesh (mesh 25 x times magnified)

The expected displacement along the face slab is shown in Figure 11. The peak deformation is occurring at elevations between 830 and 850 masl. Figure 11 shows also the results for the virtual case that Arkun would have been founded on a rock foundation. The soft foundation contributes 0.30 m deformation of the peak value of 0.45 m. Hence, the soft foundation multiplied the deformations by a factor of 2.

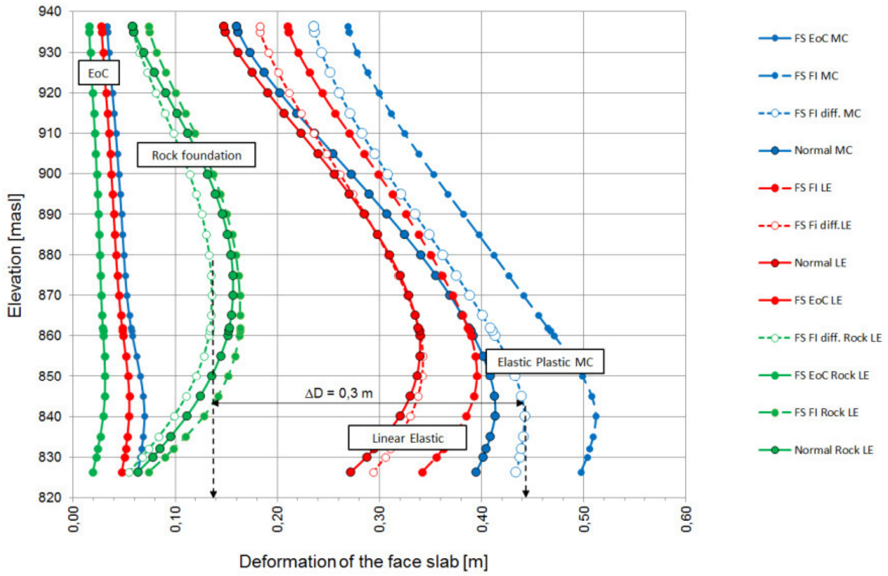

Figure 11. Predicted face slab deformation during first impoundment for the Arkun CFSGD

Benchmark data provided in ICOLD (2011) indicate a maximum deformation of the face slab of less than 0.20 m but do not distinguish between CFRDs and CFSGDs. Also the valley shape factor and 3D stress redistribution effects are not considered by the applied 2D model. In consideration also having neglected the 3D settlement effects it is expected that the predicted maximum face slab deformations are reflecting an upper value which should not be exceeded. The predicted deformations are currently analyzed in consideration of the recorded monitoring data during the first impoundment.

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