NAM NGUM 2 CFRD - BEHAVIOUR DURING CONSTRUCTION AND FIRST IMPOUNDING

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Abstract. *The 182 m high concrete face rockfill dam (CFRD) is one of the main components of the Nam Ngum 2 (NN2) hydropower scheme in Lao PDR. Construction of the dam was essentially completed in March 2010 and reservoir impounding commenced subsequently. This paper presents the main dam design features comprising dam zoning, face slab design and instrumentation. Construction principles and rockfill properties are explained. The main observations from the dam monitoring during construction and impounding are presented. The dam deformations as monitored up to date are taken as basis for back-calculations of rockfill properties. A comparison of observed dam deformation behaviour with different rockfill deformation simulation and prediction models is made.*

1 INTRODUCTION

The Nam Ngum 2 (NN2) hydropower scheme is located on the Nam Ngum river in Lao PDR, about 90 km north of the capital Vientiane and some 35 km upstream of the existing Nam Ngum 1 dam and powerhouse. With an installed capacity of 615 MW, the project will produce 2220 GWh per year energy for the Thai electricity grid. A significant component of the scheme is the 182 m high concrete face rockfill dam, with a volume of 9.7 M m^3 and a crest length of 500 m. The dam impounds a reservoir with a volume of around 4900 M m^3. A layout of the scheme is shown on Figure 1.

Construction of the NN2 Project commenced in late 2005 and was essentially completed in the second half of 2010. Rockfill placement in the dam body started in January 2008 and was completed in November 2009. Construction of the face slab, which was divided into two stages, was performed from December 2008 to July 2009 and from November 2009 to February 2010. Reservoir impounding started mid of March 2010 with the closure of the diversion tunnels. In November 2010 the reservoir had reached 97 % of the full supply level (FSL). Commissioning of the first generating unit and synchronization to the Thai grid was achieved in August 2010. Full commercial operation of the plant is scheduled to start end of December 2010.

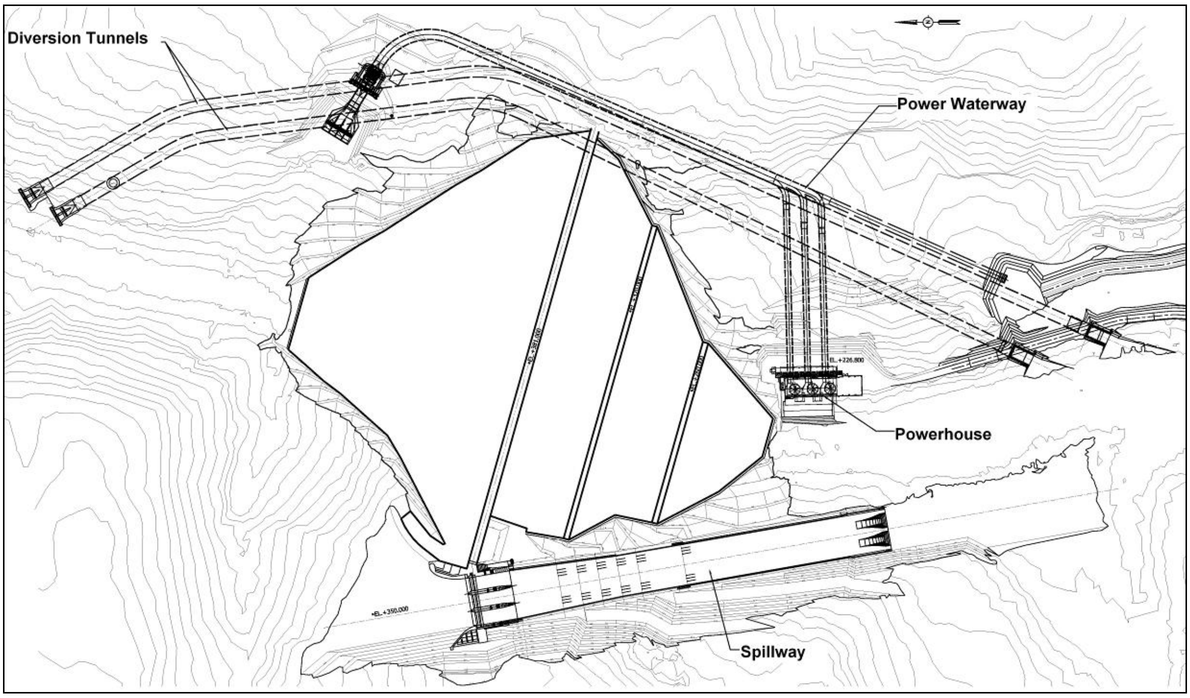

Figure 1: Layout of Nam Ngum 2 Project

2 DAM DESIGN

2.1 Geology and dam foundation

The dam is situated in a narrow valley and is founded on sedimentary rock of variable strength. The geological formations at the dam site consist of medium bedded to massive cliff-forming sandstone, interbedded with siltstone. Three easterly trending folds whose axes are nearly perpendicular to the Nam Ngum river are present at the dam site. The cliff-forming sandstone is generally slightly jointed and fractured, whereas the interbedded siltstone is moderately to closely jointed. The typical quality of the foundation rock varies within the following limits:

- Sandstone: fresh, hard and slightly fractured, locally weathered and heavily fractured.
- Siltstone: fresh to weathered, soft and slaking.

Different foundation treatment measures have been carried out to cope with the foundation rock in particular in the upstream third of the dam.

2.2 Dam geometry and zoning

The dam has a maximum height of 182 m above the foundation at the deepest section. The upstream slope is inclined at 1v : 1.4h and the downstream slope is inclined 1v : 1.4h between two berms (slope 1v : 1.5h including berms). The dam crest is 500 m long and 12 m wide. The total dam volume is 9.7 M m^3.

The dam was originally designed with a “traditional” zoning i.e. with face slab support and transition zone (2B, 3A), the main dam body of 2 rockfill zones (3B, 3C) with larger maximum size and higher lift thickness towards the downstream slope and a drainage zone (3D) at the dam bottom at the downstream 2/3 of the dam. An upstream fill (1A, 1B) is provided on the face slab.

During construction it was observed that also rockfill of moderate quality and with an increased sand content was obtained from quarrying which could not always completely be separated and wasted. Therefore, the dam zoning was adjusted to permit also placement of lower quality rockfill in the central part of the dam embankment. At the central part of the dam body the existing dense river alluvium was left in place. The final dam zoning is shown on Figure 2.

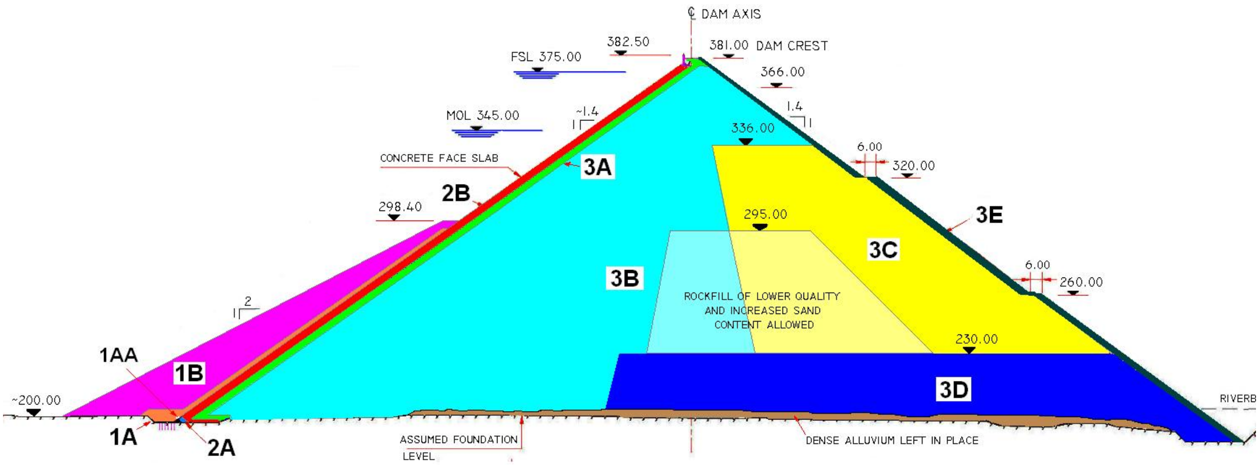

Figure 2: Dam zoning

2.3 Rockfill characteristics and construction principles

For the construction of the dam quarried rock of sedimentary formations was available. The source material, consisting basically of sandstone and siltstone, has been investigated by drilling, quarry trials and laboratory testing. Of particular interest were large scale triaxial and compressibility tests, which have been carried out by the IWHR[†] in China. From the results of these tests material parameters were derived which were then taken as basis for 2-D and 3-D deformation analysis. The results of the analyses and also visual observations indicated a high disintegration potential of the siltstone which could lead to high dam deformations. By using only sandstone for rockfill it was concluded that the dam deformation will be within acceptable and normal limits.

For the rockfill fresh to moderately weathered sandstone was used whereby the lower quality material was placed in the central zone of the dam (see Ch. 2.2).

In general the used rockfill can be characterized as follows:

- The particle size is relatively large and particle shape is angular
- The gradation is often uniform and gap-graded, with lack of gravel-size particles
- Due to the gap-grading and increased sand content, water sluicing during placement leads to the development of a mud layer on the rockfill surface which needs to be removed
- The relative densities obtained after placement were generally adequate although areas of segregated fill occasionally occurred

The rockfill of the main dam body was placed in lifts of 0.8 m (zone 3B) and 1.2 m (zone 3C) thickness and compacted with 6-8 passes of 16 t vibrating roller compactors. The amount of water added to the fill was in the range of 100 – 150 l/m^3.

[†] China Institute of Water Resources and Hydropower Research (IWHR), Department of Geotechnical Engineering

The staging of the dam construction is shown on Figure 3. Construction of the first stage of the face slab (up to EL 293.4 masl) commenced after completion of the rockfill stage 2. During the first stage face slab construction rockfill placement continued with stages 3 and 4.

The average rockfill placement rate was about 460,000 m^3/month, with a maximum placement rate of 770,000 m^3/month. The total volume of 9.7 M m^3 was placed in 21 months.

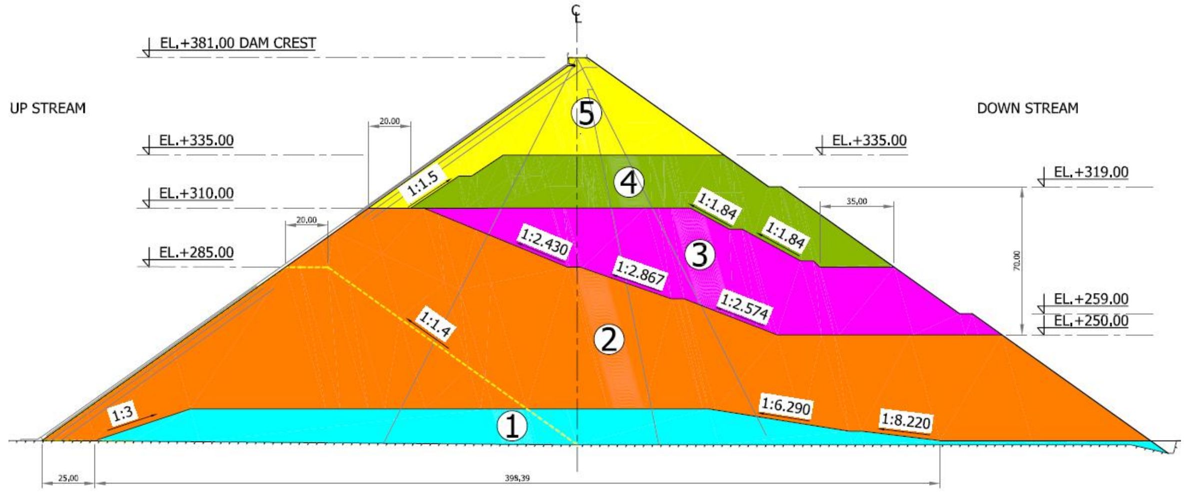

Figure 3: Stages of construction

2.4 Face slab design and construction

The dam is situated in a steep "V"-shaped valley. The concrete face slab has an area of about 88,000 m^2 and the valley shape factor is $A/H^2 = 2.7$. Considering this valley shape factor, increased movements of the face slab panels towards the river bed and resulting horizontal stresses in the central face slab panels were expected and considered in the design. Figure 4 shows the valley shape factor of NN2 CFRD plotted on the graph developed by Pinto [1].

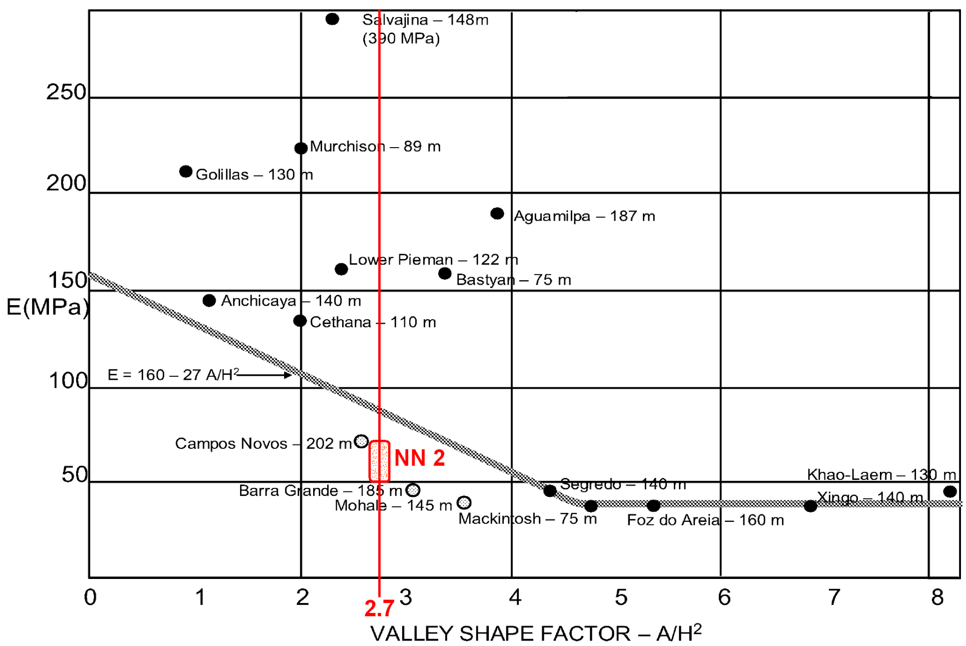

Figure 4: Graph deformation modulus vs. valley shape factor (after Pinto [1])

With a deformation modulus of 50 to 70 MPa the conditions at NN2 are about the same as at Campos Novos and Barra Grande CFRD where compression cracking of the face slab did occur.

The face slab is designed with a thickness of T = 0.3 + 0.003 H, where H is the head of water above the face slab in meters. At the upper portion of the central slabs (above EL 293.4 masl) the thickness is somewhat increased to 0.4 + 0.0018 H to improve the resistance of the slab under compression. The face slab is constructed in panels of 15 m width. At the steep abutments the slab width is reduced to 7.5 m to cope with expected increased differential settlements along the relatively steep abutment slopes.

The vertical tension joints at the abutments are designed with a bottom copper waterstop and a GB®-surface waterstop[‡], including corrugated waterstop, at the surface. An asphaltic bond breaker is applied on the concrete surface at the joint. The central joints are designed as compression joints with a 20 mm thick compressible cork filler to allow some movement of the slabs towards the valley and thus to reduce the risk of compression cracks in the slab. At the bottom a copper waterstop and at the surface a GB®-waterstop, without corrugated rubber waterstop, is provided. To maintain the designed slab thickness at the joints, the central loop of the copper waterstop is reduced and the chamfer at the slab surface is omitted. The perimeter joint is designed with copper waterstop at the bottom and GB®-waterstop system at the surface, including corrugated waterstop (Figure 5). A 20 mm thick bitumen painted wood filler is provided at the joint.

The face slab is constructed in two stages: Stage 1 up to EL 293.4 masl and stage 2 from EL 293.4 masl up to the top (connection to the parapet wall). The horizontal joint between the first and second stage of the face slab is constructed as movement joint with 20 mm joint filler board and bottom copper waterstop and surface GB®-waterstop. This joint will reduce the compressive stresses in slope direction.

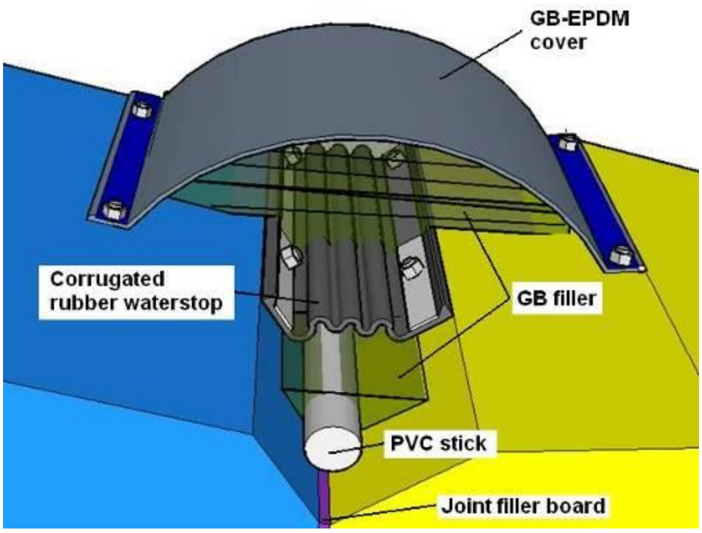

Figure 5: Schematic of GB®-Waterstop System

Double layer reinforcement of total 0.4 % in each direction is provided throughout the face slab. The reinforcement ratio is increased to 0.5 % in each direction in an area of generally

‡ GB® Waterstop Structure of Beijing IWHR-KHL Co. Ltd., China

20 m parallel to the plinth alignment and up to 40 m from the plinth at the very steep right abutment where higher stresses due to increased differential settlements are expected.

At the central slabs in an area of about 1/4 to 3/4 of the dam height, where the highest compressive forces will occur, additional stirrups are provided to prevent buckling of the upper and lower reinforcement under high compression. Anti-spalling reinforcement is provided along compression joints. The main design features of the face slab are summarized on Figure 6.

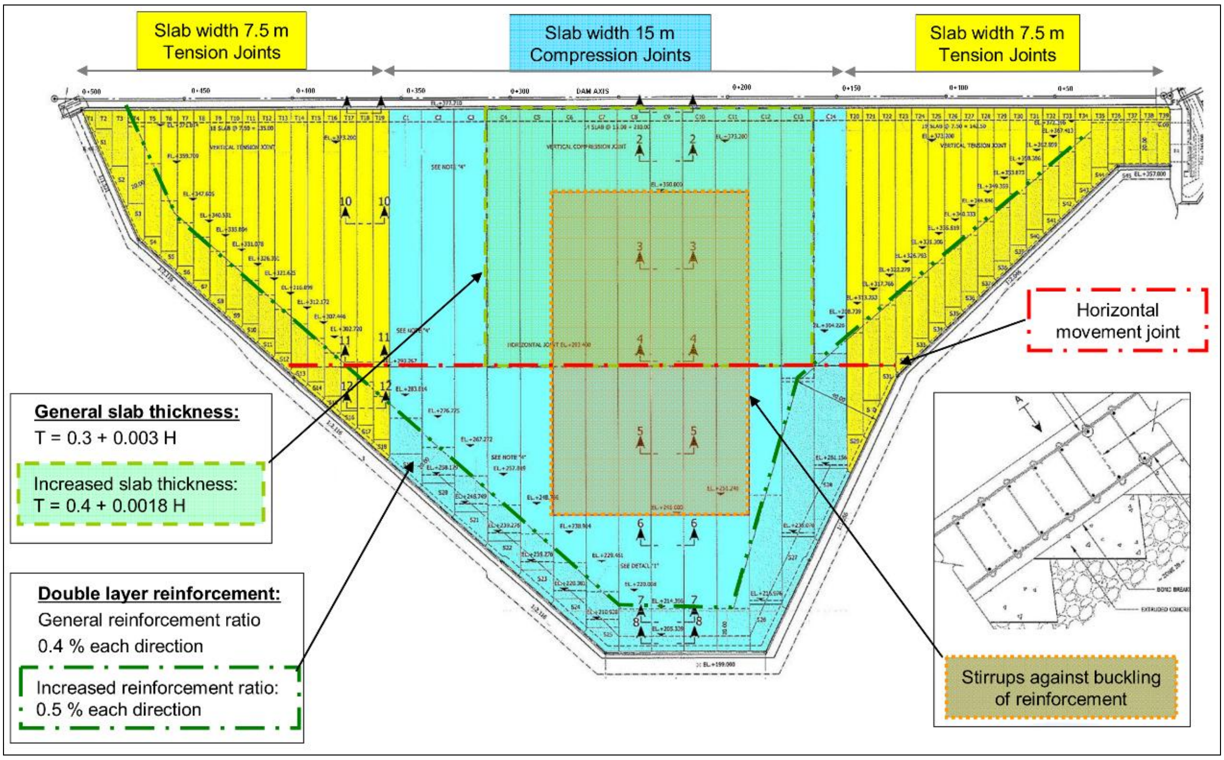

Figure 6: Face slab – main design features

For ease of construction the extruded curb method is used in the upstream face construction process. Although constructed of lean concrete of low compressive strength (about 5 MPa), the deformability of the extruded curbs is significantly lower than that of the underlain zone 2B rockfill. Therefore, if the rockfill deforms and settles the relatively stiff curbs may not always follow the movements of the fill and voids could develop during dam construction.

When the concrete face is exposed to high water load (during impounding) the curbs may crack in areas of voids. In this case the concrete slab is subject to rapid deformations and high stresses. Therefore, the development of large voids beneath the concrete curbs must be avoided or already developed voids should be filled before impounding as e.g. done at Kárahnjúkar CFRD where extensive void grouting beneath the curbs and between face slab and curbs was reportedly performed.

To reduce the development of significant voids, the curbs should be as flexible as possible to be able to move with the rockfill deformations. In this respect cutting of the curbs into smaller pieces can be considered. At some recently constructed CFRDs in China the curbs were cut vertically at the locations of the face slab joints before construction of the face slab. At NN2 grooves were excavated into the extruded curbs during its construction on both sides along the vertical face slab joints to achieve predefined break lines in the curbs. During construction it was observed that several cracks in fact developed along these grooves.

After completion of the dam rockfill construction and during the construction of the second stage face slab exploratory holes were drilled through the face slab and curbs at defined locations to investigate if voids have developed between the face slab, curbs or 2B fill. No significant voids were, however, encountered and no grouting was required.

3 INSTRUMENTATION AND MONITORING

3.1 Instrumentation concept

Instrumentation is provided to measure the behaviour of the dam during construction, reservoir impounding and long term operation. Emphasis is given towards monitoring of seepage which could arise from imperfections of the face slab and from percolations through the dam foundation, and towards monitoring of embankment deformations. A list of installed instruments is given in Table 1. The arrangement of instruments installed at the central dam section is shown on Figure 7. A plan view with the installed instruments is shown on Figure 8.

Instrument	No.	Measured Parameter
3D Joint Meters	13	Movements of face slab joints and perimeter joint
2D Joint Meters	10	Movements of face slab joints
1D Joint Meters	4	Movements of face slab joints
Sloping Inclinometer	1	Deflection of concrete face slab
Tiltmeters	23	Deflection of concrete face slab
Rebar Strain Gauges	27	Strain in the concrete face slab
Concrete Strain Gauges	27	Strain in the concrete face slab
Non Stress Strain Gauges	7	Reference strain in concrete
Magnetic Extensometers with Inclinometer Tubing	3	Transverse movement and settlements of the dam body
Hydrostatic Settlement Cells	22	Settlement of the dam body
Fixed Embankment Extensometers	113	Transverse movement of the dam body
Surface Displacement Points	35	Movements on the surface
Strong Motion Accelerometer	3	Earthquake acceleration
Total Pressure Cells	3	Total pressure at dam foundation
Distributed Fibre Optic Temperature Sensing System	1	Leakage detection along plinth (perimeter joint)
Vibrating Wire Piezometers	35	Water pressure in dam and dam foundation
Open Standpipe Piezometers	9	Ground water pressure / seepage through the abutments
Seepage Measuring Weir	1	Seepage through the dam body (concrete face slab) and abutments

Table 1: List of installed instruments

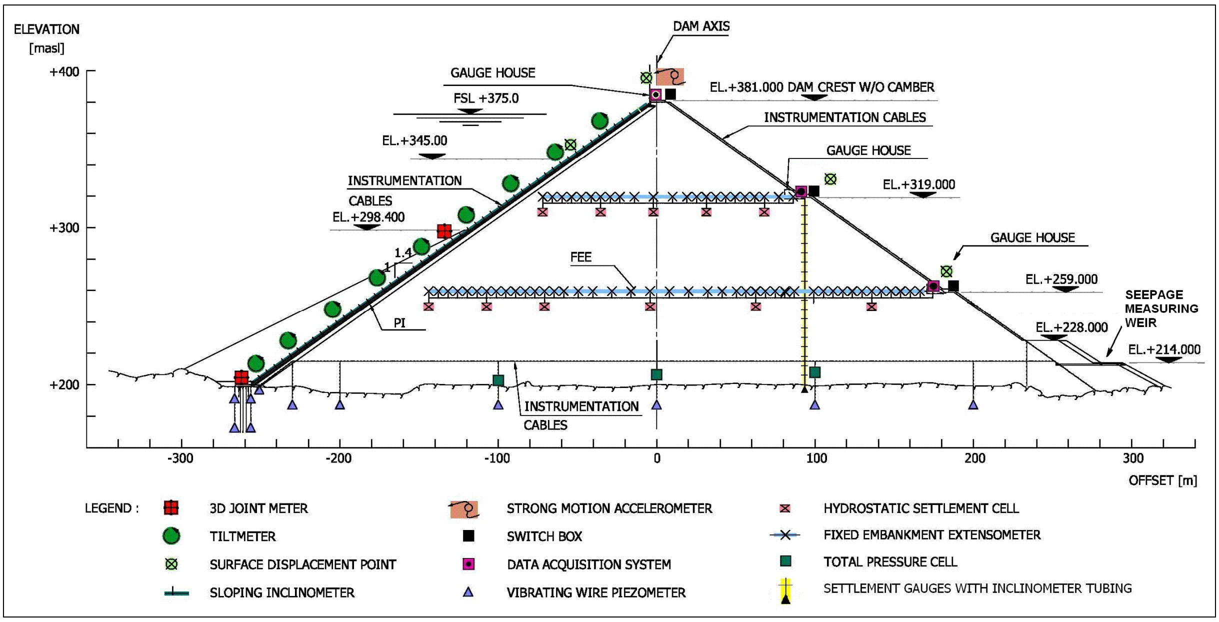

Figure 7: Arrangement of instruments at the riverbed section

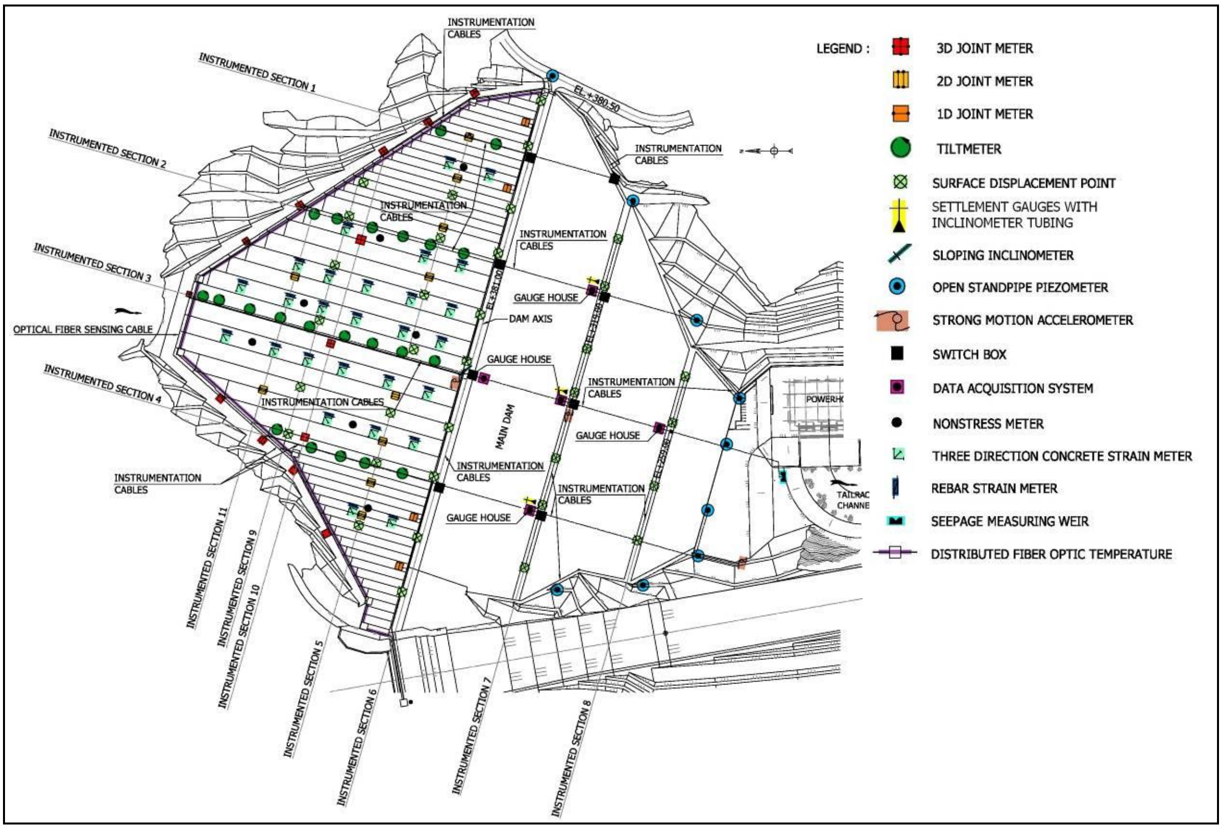

Figure 8: Overview of provided dam instrumentation

3.2 Monitoring and dam behaviour during construction

Dam deformations during construction were measured with hydrostatic settlement cells, fixed embankment extensometers and settlement gauges with combined inclinometer tubing. These instruments provided data of deformations along 3 sections within the dam body.

The maximum recorded construction settlement is around 1.8 m, measured slightly downstream of the dam centre at both instrument levels EL 260 masl and EL 319 masl. The construction settlements measured with the instruments were used for calculating the deformation moduli during construction E_{rc}:

$$E_{rc} = \frac{\gamma H d}{s} \tag{1}$$

where γ = unit weight of fill above settlement plate; H = height of fill above settlement plate; d = thickness of fill below settlement plate, s = recorded settlement of the settlement plate.

The back-calculated deformation moduli during construction are in the range of 70 MPa (upstream side) to 50 MPa (downstream side). Using these deformation moduli the dam settlements during construction were back-modelled using an elasto-plastic material model. The calculated dam settlements comply well with the measured settlement as shown on Figure 9. The maximum dam settlement at the end of construction is about 2.2 m which corresponds to 1.2% of the dam height.

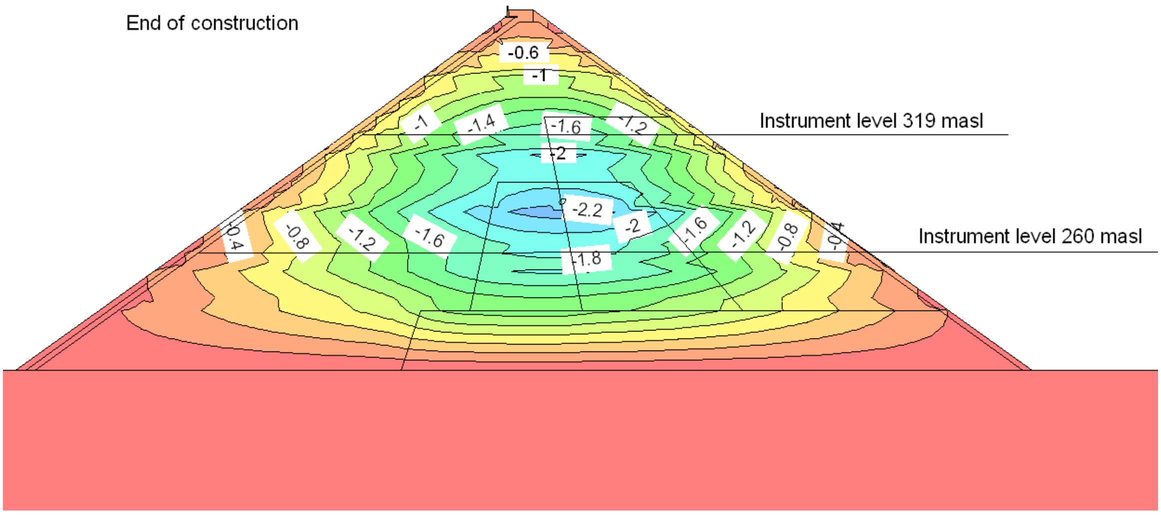

Figure 9: Dam settlements (in m) at the end of construction

3.3 Monitoring and dam behaviour during first impounding

The first reservoir impounding started in March 2010 just after the completion of the second stage face slab construction and about 4 months after completion of the dam fill. Until October 2010 the reservoir was filled up to about 95 % of the full supply level (FSL). Seepage quantities recorded with the seepage measuring weir have increased relatively linearly with the rise of the reservoir level until August 2010 when the reservoir level 350 masl was reached (approx. 85 % of FSL). Since then the recorded seepage quantity remained at 80 – 90 l/s without further increase with raising reservoir level. Investigations revealed that the maximum capacity of the seepage measuring facility was exceeded. Mitigation measures have been initiated and are still ongoing. The actually measured seepage quantity at the current reservoir level of 370 masl (97 % of FSL) is about 260 l/s.

The recorded seepage quantities until October 2010 (before rectification works at the seepage measuring structure commenced) and the reservoir fill curve are shown on Figure 10. The fluctuations of the seepage quantities are due to the heavy rainfalls during the wet season.

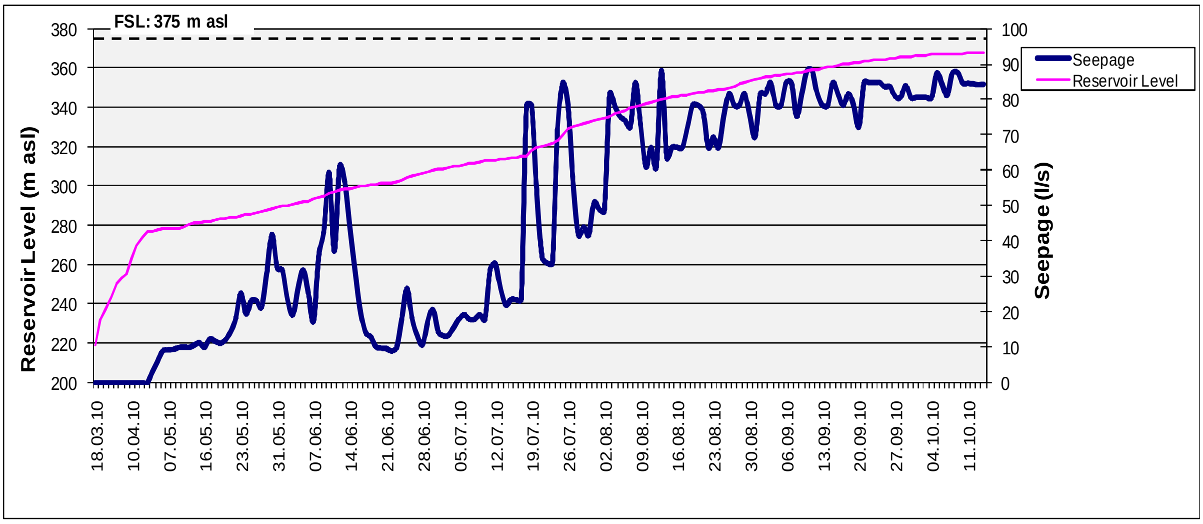

Figure 10: Seepage quantities and reservoir fill curve

The water pressures are monitored by piezometers installed in the dam and dam foundation. The piezometric data indicate a water table in the upstream third of the dam body and just downstream of the grout curtain about 30 m above the dam foundation. The water is draining towards the downstream dam toe where the water table is at the level of the seepage collecting and measuring facility. The water table in the dam body is shown on Figure 11.

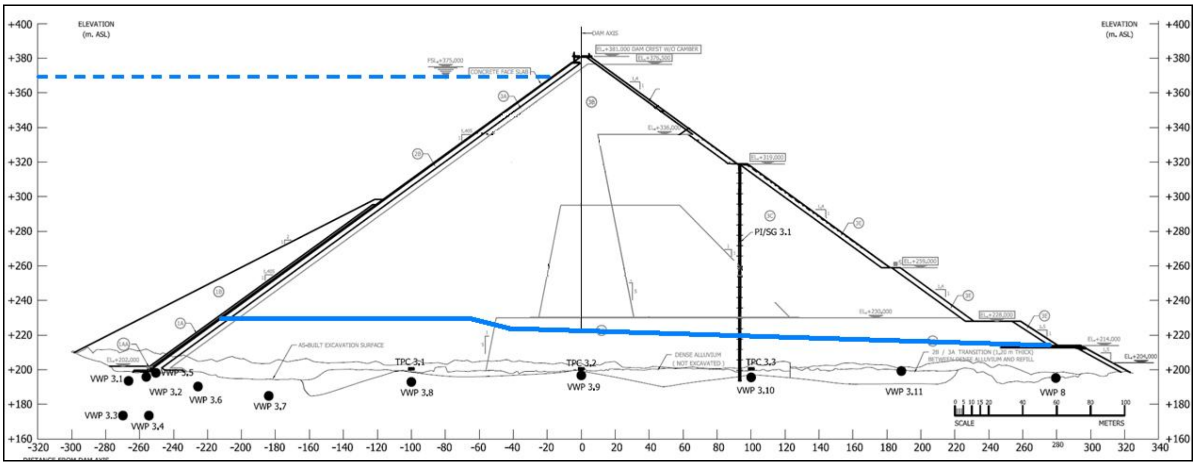

Figure 11: Water table in the dam body

Post-construction dam deformations generally occur due to the following reasons:

- Increased load from the reservoir
- Creep deformations; breakage of rock fill particles, softening of the rock fill due to wetting and saturation

The increasing water load during reservoir impounding mainly acts on the dam face and causes the concrete face and the upstream part of the rockfill to deform. The face slab deflection is measured by tiltmeters (electro-levels) installed on the face slab and by geodetic survey of surface displacement points.

The face slab deflection at different reservoir levels as measured with the tiltmeters installed at a central face slab panel is shown on Figure 12. The face slab deflection has significantly increased after the water level exceeded EL 300 masl. At a reservoir level of 369 masl (corresponding to about 97 % of FSL) the maximum face slab defection is in the order of 36 cm and occurs at about half height of the dam.

From the deformation monitoring data of the downstream side of the dam body where the reservoir load has only little or no impact quite pronounced creep settlements are observed. Data from the settlement cells in the downstream part of the dam body and geodetic survey data of the downstream shell indicate a settlement rate of 20 – 40 mm/month.

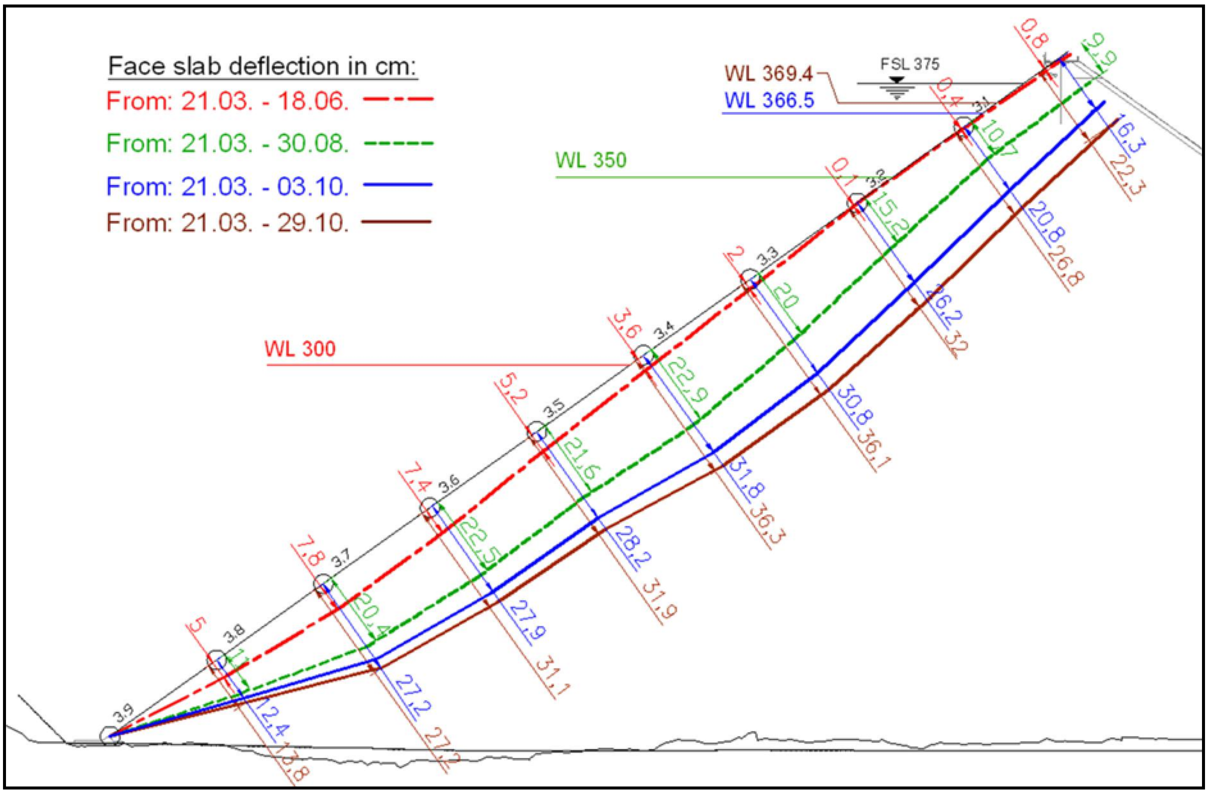

Figure 12: Face slab deflection derived from tiltmeter readings

4 DAM DEFORMATION PREDICTION AND BACK-CALCULATION

Several methods for predicting dam deformations during construction and first reservoir impounding exist. Such methods range from simplified deformation estimates based on experience, up to mathematical modeling with various constitutive models.

For the Nam Ngum 2 dam 2-dimesional and 3-dimensional numerical stress and deformation analyses have been performed by IWHR[§] prior to the construction of the dam. The analyses were performed using the non-linear hyperbolic material model. The used rockfill parameters were based on results from large scale triaxial tests carried out on samples from the quarry with gradations downscaled to suit the testing equipment. Additional calculations

[§] China Institute of Water Resources and Hydropower Research (IWHR), Department of Geotechnical Engineering have been performed using adjusted rockfill properties. The used rockfill parameters are listed in Table 2.

Materials		γ_d g/cm^3	K	K_{ur}	n	R_f	K_b	m	c kPa	φ_0 (°)	$\Delta\varphi$ (°)
Case 1	Zone 2B*	2.15	1600	3200	0.38	0.918	-0.27	2800	-	44.1	4.2
	Zone 3A*	2.15	1040	2080	0.31	0.820	-0.06	1000	-	45.9	5.7
	Zone 3B*	2.15	1000	2000	0.38	0.864	-0.29	1680	-	46.5	6.2
	Zone 3C*	2.10	630	1260	0.37	0.803	0.0	520	-	45.1	5.4
Case 2	Zone 2B	2.15	1000	2000	0.35	0.918	0.2	500	-	44.1	4.2
	Zone 3A	2.15	1000	2000	0.35	0.820	0.2	500	-	45.9	5.7
	Zone 3B	2.15	630	1260	0.37	0.864	0.1	400	-	46.5	6.2
	Zone 3C	2.10	630	1260	0.37	0.802	0.1	400	-	45.1	5.4

Table 2: Material parameters used for numerical analyses

In Table 3 the displacements obtained with the 3-dimensional analysis using the above sets of material parameters are summarized. It is noticed that the computed displacements are considerable lower than the actual displacements, indicating that the embankment materials behave softer than assumed based on triaxial testing.

Case	Case 1	Case 2
Settlement end of construction (cm)	96	140
Settlement after impounding (cm)	98	149
Face slab deflection due to impounding (cm)	20	34

Table 3: Results of 3-dimensional analyses

It should be noted that the maximum settlement at the end of construction is observed in the central downstream part of the dam which is not much influenced by the water load acting on the concrete face.

After completion of the dam construction 2-dimensional numerical analyses have been performed using deformation moduli E_{rc}, back-calculated from the settlements measured with the instruments during construction. For the analysis an elasto-plastic constitutive model was used.

As shown in chapter 3.2 the back-calculated dam deformations at the end of construction coincide well with the monitored deformations (see chapter 3.2, Fig. 9). If the same parameters would be used for calculating the face slab deflections due to impounding, a value of 1.1 m would be obtained.

However, the deformation moduli to be used for estimating face deflection are normally higher. A simple and practical way of predicting the maximum face slab deflection is to consider the deformation modulus E_T of the rockfill evaluated in the direction of the face slab movement, i.e. normal to the face slab, under the water load. Considering this transverse modulus the maximum face slab deflection D can be estimated:

$$\mathbf{D} = \frac{\gamma_\mathbf{w} \mathbf{H}_\mathbf{w} \mathbf{H}_\mathbf{d}}{\mathbf{E}_\mathbf{T}} \tag{2}$$

where γ_w = unit weight of water; H_w = height of water above face slab; H_d = thickness of fill below face slab (perpendicular to face slab), E_T = transverse rockfill modulus.

The transverse modulus E_T is higher than the vertical modulus E_V (E_V is equivalent to the modulus during construction E_{rc} as defined in Equation 1) due to a rotation of the principal stresses within the rockfill and rockfill consolidation during construction. Observed values are generally 1.5 to 5 times higher than the vertical modulus depending greatly on the valley shape factor A/H^2, but certainly also on the creep characteristics of the rockfill. Knowing the vertical modulus E_V calculated from the measured settlements during construction E_T can be estimated using the graph developed by Pinto & Marques Filho [2] (Fig. 13).

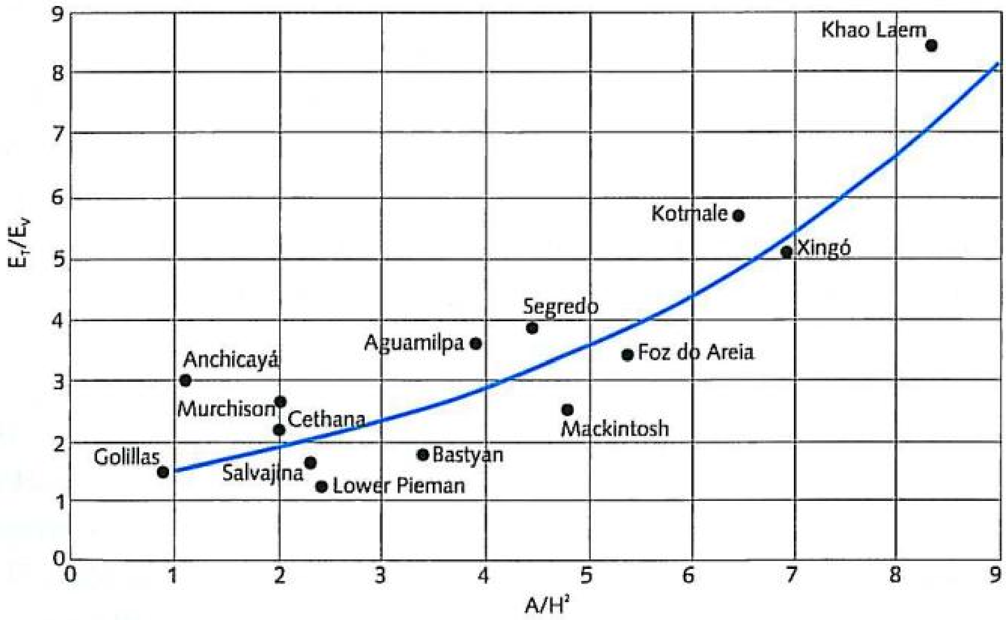

Figure 13: Ratio E_T/E_V as a function of A/H^2 [2]

Considering the valley shape factor of $A/H^2 = 2.7$ and the vertical rockfill modulus $E_V = E_{rc} = 70$ MPa the maximum face slab deflection of the NN2 dam at full reservoir level would be in the order of 65 cm occurring at about half of the dam height and full reservoir level.

5 CONCLUSION

The monitoring system of the Nam Ngum 2 dam has provided useful information on the deformation behavior of the dam during construction and first impounding. The maximum settlements at the end of construction are about 2.2 m which corresponds to 1.2 % of the dam height. The measured maximum face slab deflection due to impounding up to 95 % of the full reservoir level is 0.35 m and the maximum settlement may have reached a value corresponding to 1.5 % of the dam height. The deformations will further increase with rising reservoir to full supply level and due to creeping of the rockfill. Based on deformation monitoring at the downstream part of the dam, still a high rate of creeping is noticed.

Measured seepage quantities are at present about 260 l/s and the piezometric observations are adequate. No signs of unusual dam deformations or face slab damages are observed. At present the performance of the dam is good and the visual appearance is excellent.

The back-calculation of dam deformations and prediction of face slab deflection based on deformations measured during construction give reasonable results. A reliable estimation of the in-situ rockfill properties, in particular its creep characteristics, remains the main challenge for deformation prediction.

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[2] N.L.S. Pinto and P.L. Marques Filho, *Estimating the maximum face deflection in CFRDs*, The International Journal on Hydropower & Dams, Issue 6, 1998: 28-31.

Picture: Panoramic view of dam and reservoir