*STRESS AND DEFORMATION ANALYSIS OF THE CONCRETE FACED ROCKFILL SADDLE DAM AT GERDP AND DESIGN OF PERIPHERAL JOINT SYSTEM (*)

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1. INTRODUCTION

The Grand Ethiopian Renaissance Dam (GERD) Project is located along the Blue Nile (Abbai) almost at Ethiopian-Sudanese border, few kilometres upstream of Roseires Dam in Sudan and 700 km NE of Addis Ababa, in the Benishangul – Gumaz region.

The plant, with its 5'150MW of installed power and 15.7TWh of annual energy production, is one the most important projects in the Ethiopian

* *Analyse des contraintes et des déformations du barrage de col en enrochement de grand ethiopian renaissance dam avec noyau bitumineux et conception du système de joint périphérique*

Government’s commitment to meet the country’s present and future power requirements.

The whole project includes a roller compacted concrete (RCC) Main Dam (175m high, 10.2Mm^3 of RCC volume) and a concrete faced rockfill (CFRD) Saddle Dam (65m high, 5km long, 15Mm^3 of embankment volume). The 5’150MW installed power will be generated by 13 Francis turbines in two outdoor power-houses located at the Main Dam toe on the right and left riverside. The project also includes a nine-bay gated-spillway, an un-gated auxiliary spillway, an emergency spillway and two middle outlets to control the reservoir impounding [1]. The dam is, to date, under construction and when completed, GERD will feature the largest dam in Africa.

The Project is being implemented by the Ethiopian Electric Power company (EEP), Webuild S.p.A (former Salini-Impregilo) is the EPC contractor and Studio Pietrangeli the designer.

The paper describes the stress and deformation analysis of the Concrete Faced Rockfill dam in relation to the movements generated at the peripheral joint and at the joints between adjacent slabs. The conceptual design of the peripheral joint system and, particularly, the external waterproofing system, is then presented and discussed in the last chapter.

2. KEY CHARACTERISTICS OF THE SADDLE DAM

2.1. GEOLOGICAL SETTINGS

The Saddle Dam of GERd Project is in a contact zone where the meta-sediment (low metamorphic) collided with the basement rocks (high metamorphic). The compression stress is given by an orogenic cycle known as Mozambique Belt. The boundaries between the gneissic and volcano-sedimentary sequences are typically of tectonic origin. The geomorphology indicates that between the two dam shoulders, constituted of schist on the left side and metagranite on the right, the foundation material derives from the highly decomposed base rock.

At site scale the geological setting of the Saddle dam foundations is composed of five main geological units characterized by metamorphism at different grades of both volcanic, igneous and sedimentary rocks. The identified units are stacked by a pre-cambrian compression regime also witnessed by the presence of folds, faults and boudinage structures.

Figure 1 outlines a schematic geological plan reporting the main geological formations. In particular, the Saddle Dam is founded on rock (weathered, suffix “w” and fresh) in left and right abutments, and on residual soils (i.e. decomposed rock, suffix “d”) of variable depth in the central part.

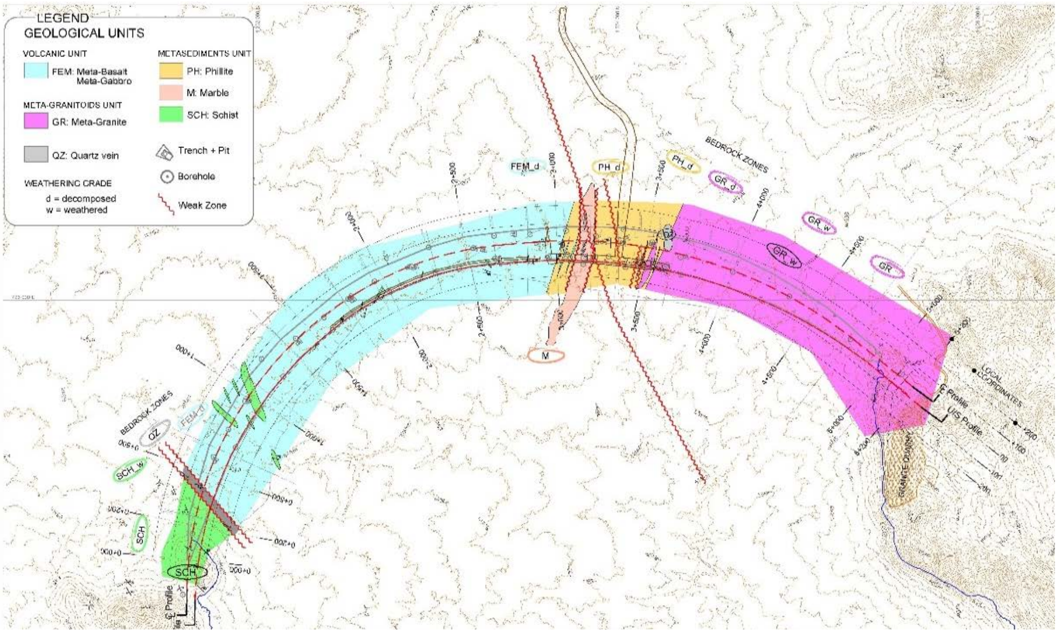

Fig. 1
Saddle Dam. Schematic Geological Plan
Barrage de col. Carte géologique schématique

From left to right bank the following five main geological units are present:

- Schist (SCH)
- Femic (FEM)
- Meta-gabbro (mG)
- Meta-basalt (mBa)
- Phyllite (PH)
- Marble (M)
- Metagranite (GR)

Minor quarzitic veins are found in the left and central portion of the foundation. Hence, the dam is founded on residual soil for a length of 3.6km and on rock for the remaining portion of 1.4km.

The boundaries between the above-mentioned geological units are often characterized by sharp tectonic contacts giving rise to the weak zones that are continuous in the upstream to downstream direction and are characterized by high permeability and potentially erodible material with different deformability characteristics. The detailed geological mapping carried out along the plinth and the inspection gallery footprint, following the progress of the excavation and concreting activities, outlined a complex geological setting made of sub-vertical slivers of material at different grades of weathering [2].

2.2. PROJECT LAYOUT

The Saddle Dam of the GERD Project is a rockfill dam with an upstream concrete facing (CFRD). Main features are summarized hereinafter:

• crest length	[m]	4’865
• maximum height (H)	[m]	65
• U/S slope	[h/v]	1.3:1
• D/S slope	[h/v]	1.3:1
• crest width	[m]	8
• Embankment volume	[Mm^3]	15
• Upstream face area	[m^2]	330’000

At full supply level the overall reservoir volume will be about 80’000 Mm^3, 65’000 Mm^3 of which will be stored above the foundation level of the Saddle Dam.

As described in the previous paragraph, the dam is founded on rock in correspondence with the abutments and in residual soil in the central portion; consequently 2 No. main typical sections have been designed:

- Section type A
  For the portion founded on rock, approximately 1’400 m long, the dam is provided with an upstream plinth and the seepage barrier is constituted by grout injections in correspondence of the U/S toe plinth;
- Section type B2
  For the portion founded on residual soil, approximately 3’600 m long, the dam is provided with an upstream inspection gallery and plinth, and the seepage barrier is constituted by composite cut-off (i.e. the combination of plastic diaphragm and grout injections) conceived to address two different requirements: permeability correction and erosion control.
  This last requirement was generally met by deepening the diaphragm down to a level of non-erodible rock or, in case of continuous at-depth potential erodible material, down to a level where the corresponding seepage gradient at the U/S toe is lower than the critical gradient [3].

Five transversal galleries allow access from downstream of the inspection gallery.

As shown in Figure 2, the embankment is subdivided, from upstream to downstream, in the following zones:

- Zone 1A: FINE BACKFILL
  Fine-graded cohesionless silt and fine sand.
- Zone 1B: COARSE BACKFILL
  Random material to provide protection to zone 1A.

- Zone 2A: PERIPHERAL JOINT FILTER
  Transition zone consisting of crushed rock particles up to 36 mm. In the event of damage of the peripheral sealing system, the filter zone 2A will prevent the movement of silt size particles through the zone.
- Zone 2B: CUSHION
  This zone provides support to the curbs and the face slab and consists of crushed sand and gravel-sized particles up to 75 mm.
- Zone 3A_1: TRANSITION 2B > 3A_2
  Transition zone 6 m wide between zone 2B and zone 3A_2, consisting of selected quarry rock with maximum size of 500 mm.
- Zone 3A_2: TRANSITION 3A_1 > 3B
  Transition zone between zone 3A_1 and zone 3B consisting of selected quarry rock with maximum size of 500 mm.
- Zone 3B: MAIN ROCKFILL
  Quarry rock particles up to 1'000 mm.
- FILTER RESIDUAL SOIL > EMBANKMENT (2B FILTER)
  A filter is foreseen above the portion of residual soil foundation between the inspection gallery and the top of the upstream trench. This filter consists of crushed sand and gravel-sized particles up to 75 mm.
- TRANSITION RESIDUAL SOIL > EMBANKMENT
  A transition layer is foreseen above the portion of residual soil between the top of the upstream trench and the D/S toe.
- DOWNSTREAM FACE
  Large blocks are dozed to the downstream face of the dam to protect the downstream face and to realize a toe with a gentler slope of 1.6:1 (h/v).

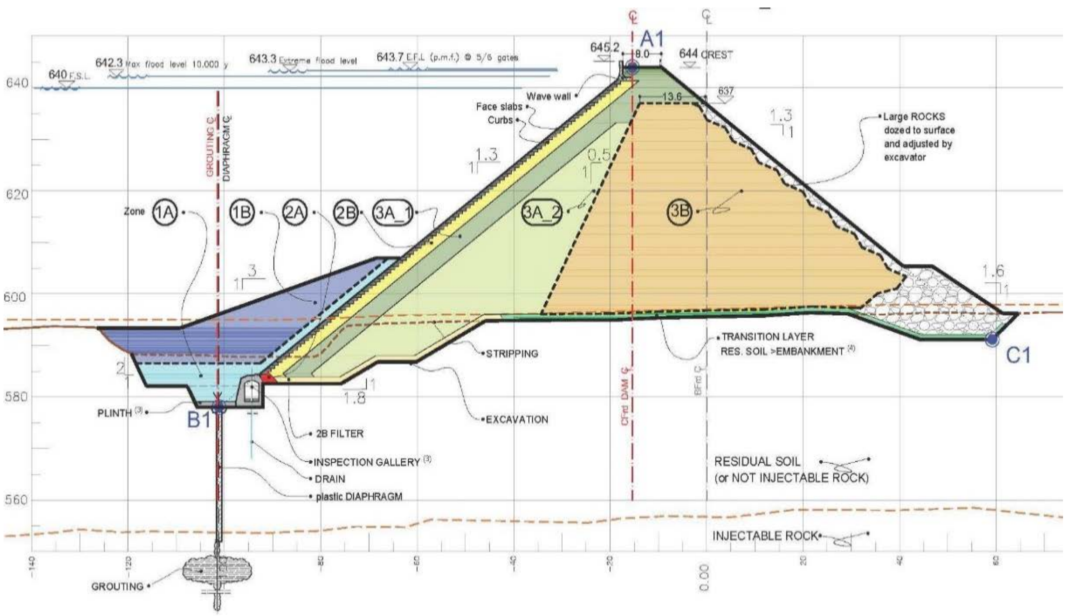

Fig. 2
Saddle Dam. Typical section on residual soil
Barrage de col. Coupe type sur terrain meuble

2.3. CONCRETE FACE SLAB

The primary water barrier of the CFRD is constituted by the reinforced concrete face slabs poured on the underlying support zones, represented schematically in Figure 3.

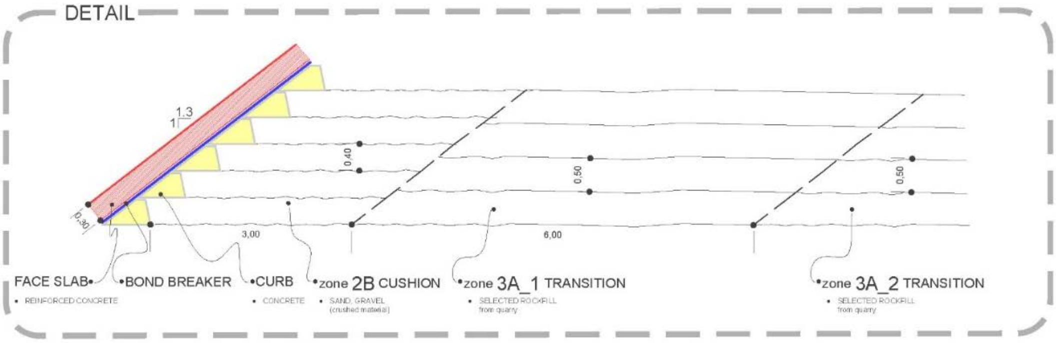

Fig. 3
Zoning Detail at the face slab
Détail du zonage de la dalle de face du barrage en remblai

The support zone includes:

- extruded concrete curb (0.4 m thickness),
- bond breaker (PVC sheets) between curbs and slabs to prevent bond and consequently reduce tensions in the concrete slab induced by rockfill deformation under reservoir water load.

The polycentric shape of the Dam centreline has been discretized with segments 50m long therefore the dam U/S face results subdivided in a series of planar elements with a trapezoidal shape.

To simplify and homogenize the geometry of the slabs each trapezoidal element has been subdivided in No. 3 slabs (12.1m wide) and a variable “compensation” slab with a trapezoidal shape. Locally the spacing of the slab joints has been adjusted to better cope with the geometry of the plinth/gallery and of the geometry/quality of the foundation.

Starter slabs have been locally introduced where the slope of the plinth is such as to prevent the use of the slip-formworks from the peripheral joint.

Details of the reinforced concrete slab are summarized hereafter:

• Tk	[m]	0,30	Thickness of the face slab
• Wb	[m]	12.1	Width of the slab (predominant)
• C	[-]	C20D20	Concrete class
• R	[-]	1 layer Φ16@15	Reinforcement bars (0.45% of slab cross section area, both vertical and horizontal directions)
• c	[cm]	17	Reinforcement cover

3. STRESS AND DEFORMATION ANALYSIS

3.1. GENERAL

The main objectives of the stress and deformation analysis are:

- study of the deflection of the concrete slab under reservoir loading due to the deformation of the underling rockfill body in the cross section-plane and verification of the reinforcement bars (2D analysis);
- estimate of the peripheral joint movement under static and dynamic loadings in the cross-section plane (2D analysis);
- identification of the zones of dam face in compression or in tension (3D analysis).

The stress and deformation analyses described in this chapter are focused on the last two points.

As far as the reinforced concrete verification is concerned, it is noted that the slabs are supported by the underlying rockfill body and are not required to resist any transversal loading. In general, at the scale of the dam, the rockfill deformations are small leading to a large radius of curvature of the slab deformed-shape and negligible bending stresses. Therefore, the design of the slabs is governed by the necessity of maintaining their water-tightness under the deformations of the embankment, ensuring their durability (and therefore adequate protection to the reinforcement). Both the above requirements are satisfied by designing the reinforcement bars to limit the concrete crack openings below an assigned value.

3.2. TWO-DIMENSIONAL ANALYSIS

The two-dimensional analyses have been carried out by means of finite element modelling (Phase2 version 8 by Rocscience and Midas GTS NX) and include both Static analysis and Dynamic analysis.

The analyses presented in this paragraph are relevant to section Type B2 at chainage ch . 3+300 (PH_d), which is the most critical section for the settlement analysis of the portion of the Saddle Dam founded on residual soils (maximum height of the dam and maximum thickness of decomposed soil).

As far as the dynamic analysis is concerned, two types of analysis have been carried out:

- Modal analysis with Response Spectrum Analysis (RSA);
- Time History analysis (TH).

In a modal analysis with Response Spectrum, modal dynamic properties are evaluated first and then the response of each individual mode using a Response Spectrum. Effects of the different modes are combined using the Complete Quadratic Combination rule, which is an approximate, probabilistic based rule and introduces loss of the sign of the computed effects.

The modal analysis is performed on the same Finite Element model used for the static analyses with masses of all components derived from material densities. Boundary conditions are the same as for the static analysis. Additional masses for water interaction have been derived based on Westergaard theory (as is normal practice).

The time-history analysis has been used mainly for validation of the RSA and therefore one single input signal has been used. The solution has been implemented using a modal integration technique including the first seven modes and assuming a modal damping equal to 10% of the critical damping. In this method, a discrete time step integration is numerically carried out for each mode and the total response is obtained by mode superposition. The technique exploits the computational advantages of dealing with uncoupled equations and permits a reduction in the number of relevant degrees of freedom (by not including higher modes above a threshold of engineering interest) without compromising accuracy.

Deformability characteristics of dam and foundation materials (at section 3+300) used in the numerical model are summarized in the following figure.

		Z(1) [m]	γ [kN/m^3]	E_{rc}(2) [MPa]	E_{rf}(2) [MPa]
FOUNDATION	Top soil	<2	19.5	50	
	PH residual soil	2÷4	19.5	160	
	PH residual soil	4÷6	20.5	200	
	PH residual soil	6÷8	20.5	240	
	PH residual soil	8÷10	20.5	280	
	PH residual soil	10÷12	20.5	320	
	PH residual soil	>12	20.5	400	
EMBANKMENT	2 A rockfill	-	23.0	100	150
	2 B rockfill + 2B filter	-	23.0	100	150
	3A_1 rockfill	-	22.0	80	100
	3A_2 rockfill	-	21.0	60	80
	3 B rockfill	-	20.0	40	60
	3 C rockfill	-	17.0	40	60
	TRANSITION	-	22	80	100
	1A FINE Backfill	-	18.0	100	100
	1A COARSE Backfill	-	18.0	100	100
	CF	-	24.5	31200	
	Plinth / Inspection Gallery	-	24.5	31200	

NOTES:

(1) Depth from original ground

(2) For the embankment materials two values of modulus are reported: Modulus during construction (Erc) and modulus during the full supply level (Erf).

(3) The backfill is reported in the figure only for illustrative purpose, in the FEM model this materials is considered as external load.

Fig. 4

Deformability characteristics of dam and foundation materials

Paramètres de déformabilité du barrage et des matériaux de fondation

Results of the 2D static analysis for section 3+300 under reservoir load are shown in the following figures where it is possible to observe that:

- maximum total displacement of concrete slab is equal to about 13cm and is located at about one third of the embankment height
- displacement in correspondence of the gallery foundation is equal to about 7-8cm (3cm in horizontal direction, 6-7cm in vertical direction).

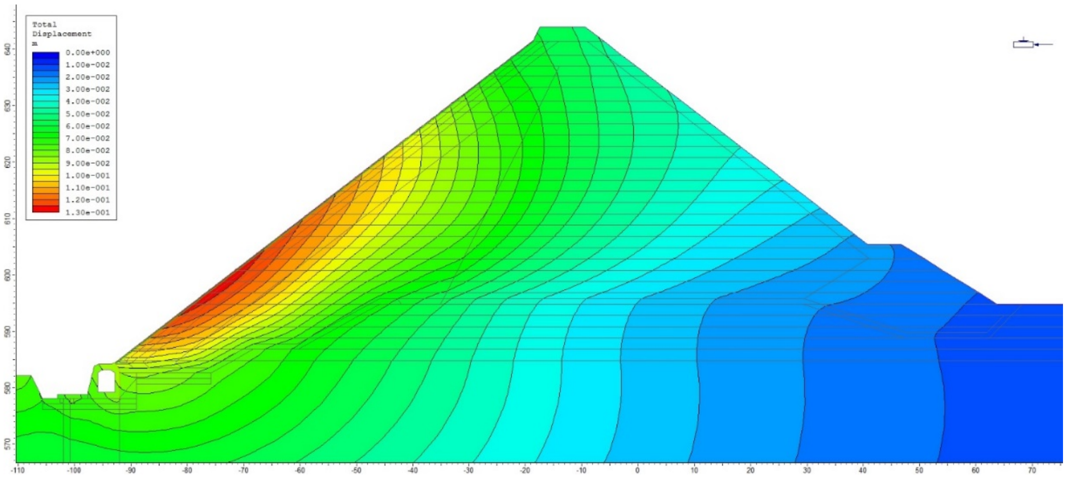

Fig. 5
Section 3+300 (type B2), FSL, Total displacement contour map
Section 3 + 300 (type B2), FSL, Carte de contour de déplacement total

The relative movements between plinth and slab are equal to about:

- ~1.0cm bridging
- ~0.5cm sliding

substantially in line with the reference values reported in literature (see Figure 6).

Additional analysis has been carried varying both the elastic modulus of rockfill and foundation to study the influence of these parameters on the slab deflection, opening of peripheral joint and differential settlement. In particular, the following conservative hypothesis have been analysed:

- Rigid Foundation
  Elastic modulus of foundation is increased from 400MPa to 20'000MPa to simulate a more rigid zone in the residual soil foundation (e.g. marble area) and increase the relative movement between embankment and gallery (the D/S face of the gallery is in fact poured directly against the upstream excavated trench).
- Rockfill compaction anomaly
  Elastic modulus of the rockfill underlying the concrete slab is reduced to one third of the design value to simulate a local compaction anomaly in the embankment.

Results, in terms of peripheral joint bridging, are reported in the following figure where:

- the RED dot corresponds to the reference calculation scenario
- the GREEN dot refers to the sensitivity analysis with rigid foundation and rockfill anomaly
- the GREY dots correspond to reference projects reported in ICOLD bulletin No. 141 [4]

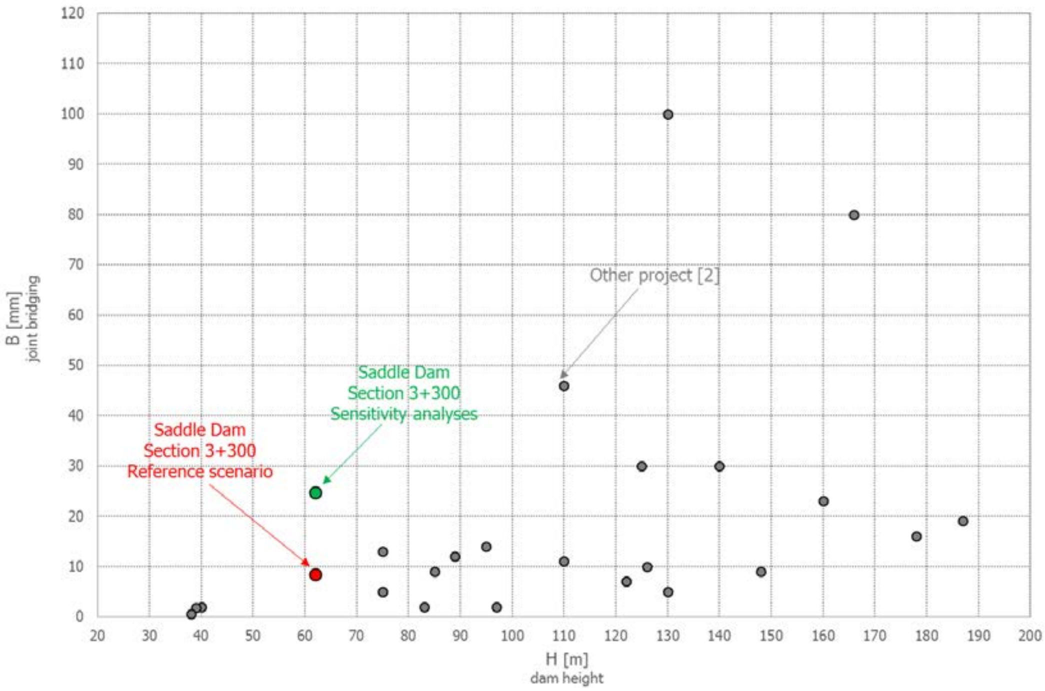

Fig. 6

Section 3+300 (type B2), FSL, Peripheral joint bridging

Section 3 + 300 (type B2), FSL, Ouverture du joint périphérique

The presence of several sub-vertical contacts nearly orthogonal to the dam axis with sharp changes in the foundation moduli, may produce a differential sliding of the slab-to-slab joints up to 8cm.

The seismic displacements under SEE are reported graphically in the following figure which shows displacement contours of the whole dam.

Same displacement calculations have been carried out also through direct integration of Time Histories.

It is observed that:

- the two methods (RSA vs TH) are well consistent.
- total additional displacement due to earthquake (at perimetral joint) is in the range of 12mm.
- relative additional displacement due to earthquake (at perimetral joint) is in the range of a millimetre.

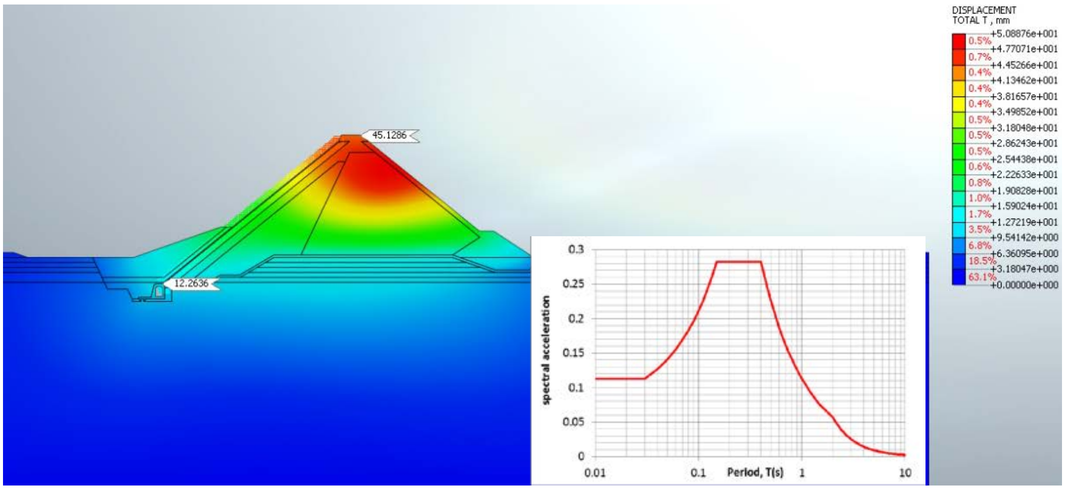

Fig. 7
Section 3+300 (type B2), Total displacement under SEE and SEE spectrum
Déplacement total sous charge sismique SEE et spectre de réponse SEE

3.3. THREE-DIMENSIONAL ANALYSIS

The purpose of 3D analysis is to identify zones where the slabs are expected to be horizontally in compression and zones where they are expected to be horizontally in tension. Anti-spalling reinforcement should be provided at the joints where significant compressions are foreseen.

Since the Saddle dam presents a very large valley shape factor ($A / H^2 > 50$) no compression zones are foreseen, except for localized area near the abutments or in correspondence of geometrical irregularities of the foundation.

Three partial three-dimensional models have been generated using Midas GTS NX software, to investigate the 3D effects at the dam surface, in relation to the face slab behaviour (right abutment left abutment, central portion of the dam).

As for the two-dimensional model, boundary conditions assume fixed displacements at the base of the volume modelled and horizontal restraint on the vertical faces.

Gravity loading and water have been applied in stages as for the two-dimensional analysis to isolate the strains at the slab face due to the reservoir loading (which are the only component of interest in this study).

The following figure shows an isometric view of the total dam displacements under reservoir loading (right bank).

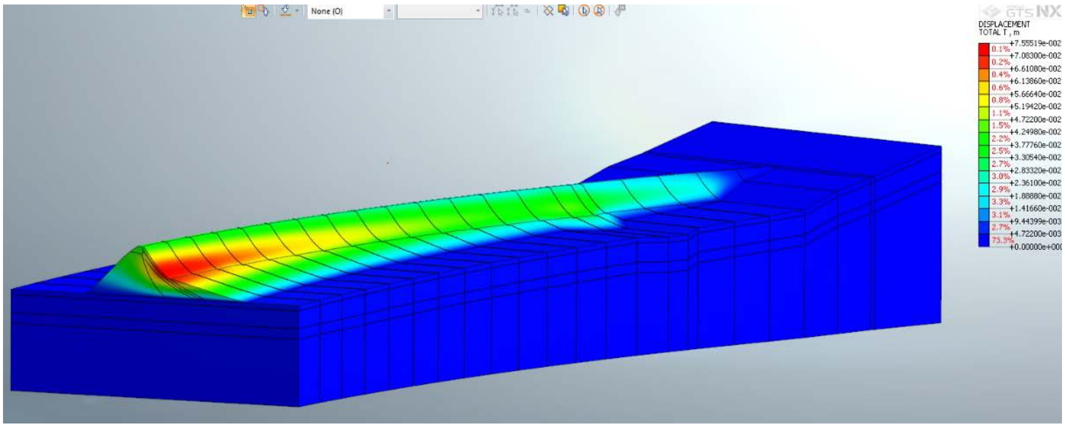

Fig. 8
Total displacement under reservoir loading (right bank model)
Déplacement total du barrage sous charge hydraulique du réservoir (modèle de l'appui droit)

The following diagram shows in isometric view the compressive strains measured along horizontal fibres of the dam face (Eyy). Zones where contours are not displayed are tensile zones.

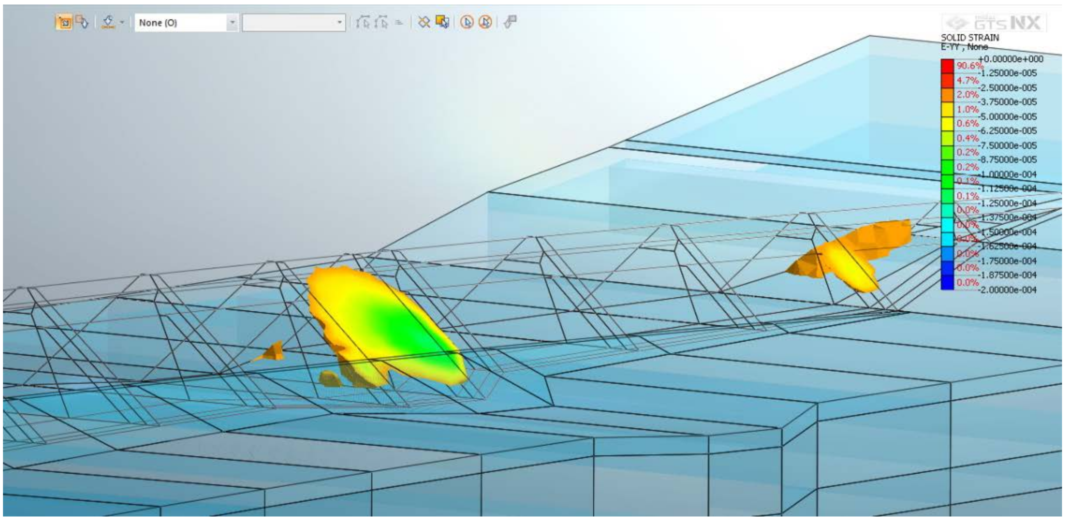

Fig. 9
Isometric view of Eyy compressive strains under reservoir loading
Vue isométrique des déformations compressives d'Eyy sous charge hydraulique du réservoir

It is observed that compression is limited to the areas at the bottom of the right bank and in correspondence of a foundation slope change located between ch. 4+680 and 4+740.

The other portions of the dam face are almost neutral or under low tension.

Maximum compressive strains are estimated in the order of E-4 corresponding approximately to a compressive stress of 2-3MPa and a closure of vertical joint of about 1mm.

In view of the results of the 3D analysis no additional anti-spalling slab reinforcement has been designed. For the sake of safety, a compressive joint has been adopted for the vertical slab joints that lie within the compressive zones.

In the other portions of the dam face (where tension is observed) the maximum calculated opening of vertical joints is in the range of few millimetres.

4. PERIPHERAL JOINT SYSTEM

The peripheral joint system, connecting the concrete face slabs and the plinth/gallery of the CFRD, must guarantee the water tightness of the dam against reservoir load while allowing for the anticipated movements between plinth / gallery and face slabs [4].

The perimeter joint of the Saddle Dam is constituted by a double system: an internal copper waterstop and an external waterstop system.

The two systems have been designed so that they can accommodate, without being damaged, the relative movements between face slabs and plinth / gallery, with adequate margins to cover intrinsic inaccuracies of the numerical model and possible unforeseen local weakness/defects of embankment and foundation.

The conceptual design of the external waterstop system, which is the object of this paragraph, foresees two different elements with specific functions:

- Structural
  The structural function (bearing of the water load over the joint opening) is carried out by a mono-directional carbon-fibre layer.
  This material includes an 85gr/m^2 carbon-fibre warp yarn and an 85gr/m^2 fibre-glass weft yarn (Mapenet Hybrid 85/100)
- Waterproofing
  The waterproofing function is guaranteed by a polyvinyl-chloride plasticized geo-membrane with a thickness of mm (Mapeplan WTS 40).

Both materials have been supplied by *Mapei* who carried out, in coordination with *Studio Pietrangeli*, a testing program and 1:1 scale joint prototypes in their laboratory in Milan and on site. These prototypes allowed the fine-tuning of the complex 3D geometry of the joint, the optimization of the shape of materials and the fastening details to the concrete structures, in particular at the crossing with vertical joints.

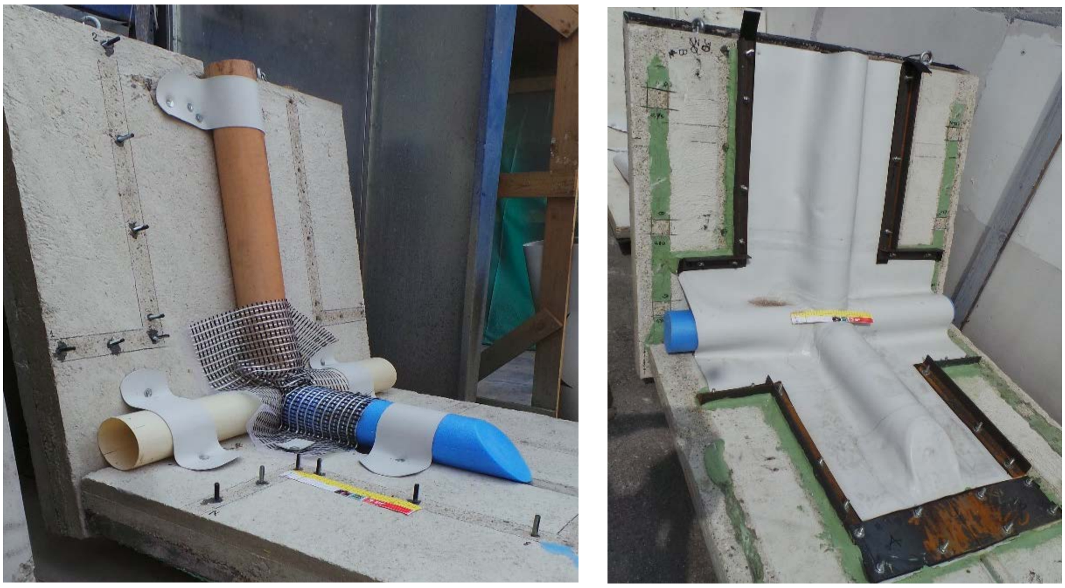

Fig. 10
Prototype of the external waterstop system (*Mapei*, Milan)
Prototype du système « waterstop » externe (Mapei, Milan)

Preliminary tests carried out at *Mapei* laboratory indicated that the adopted PVC geomembrane shows a progressive deformation at constant load (Figure 11). This behaviour becomes much more evident as the testing temperature increases and could lead to a progressive failure of the membrane, even if the initial elongation is well below the ultimate one. It has been therefore necessary to introduce a material with a structural function in order to keep substantially unloaded the membrane.

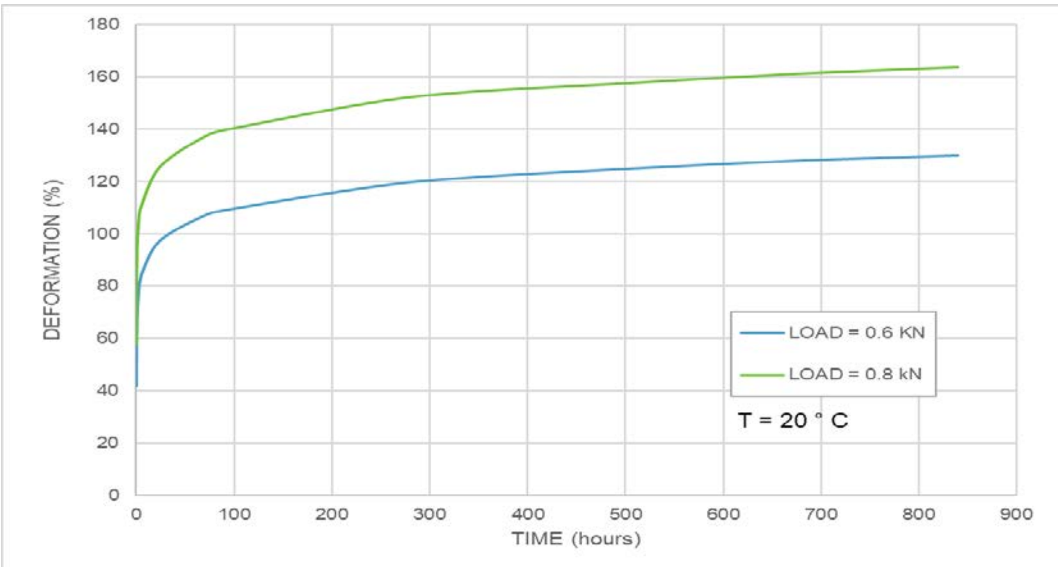

Fig. 11
PVC geomembrane, Mapeplan WTS 40 (4mm), creep curve at 20 °C
Géomembrane en PVC, Mapeplan WTS 40 (4mm), courbe de fluage à 20° C

The adopted 85gr/m^2 carbon fibre structural grid (Mapenet Hybrid 85/100) has an ultimate elongation of about 2% (i.e. much lower than that of the PVC geomembrane which is about 320%) and a modulus of 245GPa (i.e. much larger than that of the PVC geomembrane). Consequently, the high stiffness material, once loaded, develops minimal elongations that can be followed by the PVC geomembrane without developing significant tensions, thus preventing its water tightness function.

The external waterstop system is fastened at its periphery to the concrete structures by means of stainless-steel profiles placed over an epoxy resin bedding and threaded bars chemically anchored to the concrete structures and fastened in order to guarantee that the periphery of the external waterstop system is in compression all along its length.

The length of carbon grid and PVC membrane, based on the predicted joint movements, is accommodated by a profile constituted by folded polypropylene geotextile. Under the pressure of water this material is forced into the opening resulting from the movements of the joints, supporting the membrane and giving, in case of leakages, an extra-line of defense by clogging the opening and limiting the inflow of water (in nautical terminology it is known as "*collision mat*").

The carbon-fiber structural grid has been tested in the Department of Structures for Engineering of the University of Naples "Federico II". The tests have been carried out according to ASTM D 3039 "Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials" in one or double sheet configuration using specimens made up of 2 or 4 threads.

The tests were conducted in displacement control with a constant head-speed of 2mm/min. Aluminum tabs were connected to the specimens ends in order to assure appropriate and adequate stress distribution in the gripping area which was kept constant during testing at 14MPa. Test results are summarized in the following figure:

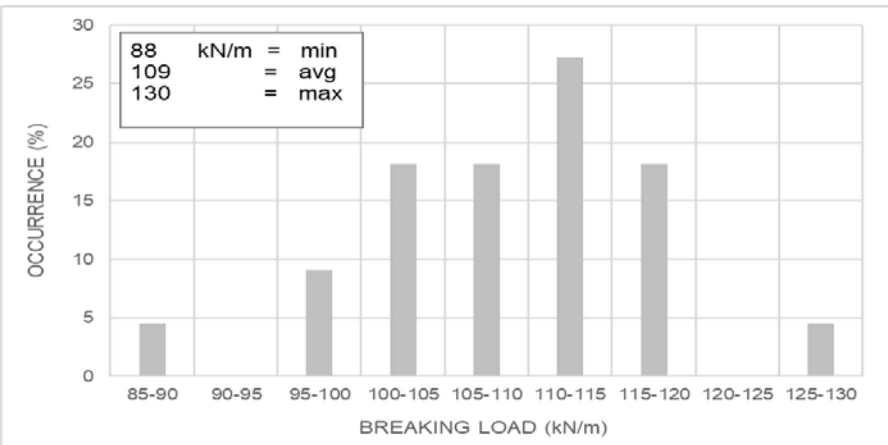

Fig. 12

Summary of Carbon grid tensile test results, Mapenet Hybrid 85/100

Résumé des résultats des tests de traction sur les treillis en fibre de carbone

A simple calculation carried out with Mariotte formulation indicates that the carbon grid is capable of bearing the maximum hydrostatic load of 60m over a 30cm wide opening, which is more than adequate for the requirements of the peripheral joint of the Saddle dam.

5. CONCLUSIONS

This paper discusses the main design aspects pertaining to the waterproofing of the Saddle Dam of the Grand Ethiopian Renaissance Dam Project.
Waterproofing is ensured by a reinforced concrete facing and by the peripheral joint system connecting the face slabs and the plinth/gallery.

The design of both the above elements is largely influenced by the deformation of the dam under water and seismic loading.

The analytical work carried out for the estimate of the possible displacements (consisting of two- and three-dimensional modelling) has shown that they are in line with those reported in technical literature. The numerical calculations included sensitivity analysis on key parameters (such as stiffnesses of dam and foundation materials) and allowed the identification of more conservative scenarios for the design of the waterproofing system, which is discussed in the last part of this paper.

In particular, the conceptual design of the external waterstop system foresees two different materials with specific functions:

- the STRUCTURAL function is guaranteed by a mono-direction carbon-fibre layer (85gr/m^2);
- WATERPROOFING is guaranteed by a polyvinyl-chloride plasticized geo-membrane with a thickness of 4mm.

The elevated stiffness carbon-fibre grid, once loaded, develops minimal elongations that can be followed by the PVC geomembrane without developing significant tensions, thus preserving and ensuring its water tightness function.

The length over the joint of the carbon grid and PVC membrane, based on the predicted joint movements, is supported by a profile constituted by folded polypropylene geotextile. This material is forced into the joint opening by the reservoir pressure and, in case of leakages, provides an extra-line of defense by clogging the opening and limiting the inflow of water (in nautical terminology it is known as “*collision mat*”).

As illustrated in the paper, the system is well suited to accommodate the predicted deformation and is also resilient to local defects. The detail of this system has been developed through a specific campaign of testing and full-scale prototypes.

REFERENCES

[1] PIETRANGELI, G., BEZZI, A., ROSSINI, C., MASCIOTTA, A., D'ALBERTI D., (2017). *Design of Gram Ethiopian Renaissance RCC Main Dam (H = 175m).* Water Storage & Hydropower development for Africa.

[2] PIETRANGELI, G., CACCIARINI, A., CALABRESE, M., (2013). *Geo-mechanical Characterization of residual soils for foundation design of a Bituminous Faced Rockfill Dam.* Water Storage & Hydropower development for Africa.

[3] PIETRANGELI, G., PITTALIS, G., RINALDI, M., CIFRA, R., (2019). *Design and construction of composite cut-off system at Grand Ethiopian Renaissance Dam, Ethiopia.* Proceedings of World Tunneling Congress.

[4] ICOLD (2010). *Concrete Face Rockfill Dams*. Bulletin 141. Paris: International Commission on Large Dams.

SUMMARY AND KEY-WORDS

The Grand Ethiopian Renaissance Dam (GERD) Project is located along the Blue Nile (Abbai) almost at Ethiopian-Sudanese border, a few kilometres upstream of Roseires Dam in Sudan and 700km NE of Addis Ababa.

The plant, with its 5'150MW of installed power and 15.7TWh of annual energy production, is one the most important projects in the Ethiopian Government's commitment to meet the country's present and future power requirement, once completed GERD will feature the largest dam in Africa.

The whole project includes a roller compacted concrete (RCC) Main Dam (175m high, 10.2Mm^3 of RCC volume) and a concrete faced rockfill (CFRD) Saddle Dam (65m high, 5km long, 15Mm^3 of embankment volume).

At site scale the geological setting of the Saddle dam foundations is composed of five main geological units characterized by metamorphism at different grades of both volcanic, igneous and sedimentary rocks. In particular, the Saddle dam is founded on rock in left and right abutments (for an overall length of 1.4km) and on residual soils of variable depth in the central part (for an overall length of 3.6km). The boundaries between these geological units are often characterized by sharp and sub-vertical tectonic contacts giving rise to the weak zones that are continuous in the upstream to downstream direction and are characterized by high permeability and potentially erodible material with different deformability characteristics.

The waterproofing of the Saddle Dam is ensured by a reinforced concrete facing poured on the underlying curbs and by the peripheral joint system connecting the face slabs and the plinth/gallery constituted by an internal copper waterstop and an external waterstop system. The design of both the above elements is largely influenced by the deformations of the dam which has been analysed by means of 2D and 3D numerical models.

The analytical work has shown that the predicted deformations are in line with those reported in technical literature. The numerical calculations included sensitivity analysis on key parameters (such as stiffnesses of dam and foundation materials) and allowed the identification of more conservative scenarios for the design of the waterproofing system. In particular, the conceptual design of the external waterstop system foresees two different materials with specific functions:

- STRUCTURAL function is guaranteed by a mono-direction carbon-fibre layer;
- WATERPROOFING is guaranteed by a PVC plasticized geo-membrane.

The high stiffness carbon-fibre grid, once loaded, develops minimal elongations that can be followed by the PVC geomembrane without developing significant tensions, thus preserving and ensuring its water tightness function.

The length over the joint of the carbon grid and PVC membrane, based on the predicted joint movements, is supported by a profile constituted by folded polypropylene geotextile. This material is forced into the joint opening by the reservoir pressure and, in case of leakages, provides an extra line of defense by clogging the opening and limiting the inflow of water (in nautical terminology it is known as “*collision mat*”).

The peripheral joint system is well suited for accommodating the predicted deformation and is also resilient to local defects. The detail of this system has been developed through a specific campaign of testing and full-scale prototypes carried out by *Mapei* in coordination with *Studio Pietrangeli*.

Le projet du Grand Ethiopian Renaissance Dam (GERD) se situe le long du Nil Bleu (Abbai) à proximité de la frontière entre l’Ethiopie et le Soudan, à quelques kilomètres à l’amont du Barrage de Roseires au Soudan et à 700 km au nord-est d’Addis Abeba.

Avec ses 5’150MW de puissance installée et sa production annuelle d’énergie de 15,7TWh, cette centrale hydroélectrique représente un des projets les plus importants du Gouvernement de l’Ethiopie pour faire face aux besoins d’énergie présents et futurs du pays. Une fois la construction terminée, le projet GERD inclura le barrage le plus grand d’Afrique.

Dans sa globalité, le projet comprend un barrage principal en béton compacté au rouleau (BCR) de 175m d’hauteur et 10,2Mm^3 de volume et un barrage de col en enrochement avec noyau bitumineux protégé par des dalles en béton (hauteur du barrage 65m, longueur 5km, volume 15Mm^3).

A l'échelle du site, la géologie de la fondation du barrage de col est composée de cinq unités géologiques principales caractérisées par des différents degrés de métamorphisme de roches volcaniques effusives et intrusives et roches sédimentaires. Les appuis gauche et droit du barrage de col sont fondés en roche (pour une longueur totale de 1.4 km) tandis que la partie centrale est fondée sur des sols résiduels de profondeur variable (pour une longueur totale de 3.6 km). Les limites entre les différentes unités géologiques sont souvent caractérisées par des contacts tectoniques aigus et sub-verticaux qui produisent des zones de faiblesse continues dans la direction amont-aval et qui se distinguent par une perméabilité élevée et par une forte tendance à l'érosion avec des différentes caractéristiques de déformabilité des matériaux.

Le système d'étanchéité du barrage de col est composé d'un masque amont en béton renforcé sur les bordures sous-jacentes et par un système périphérique de joints reliant les dalles de face avec le socle / galerie d'inspection constitué par un « waterstop » interne en cuivre et par un système « waterstop » externe. Le dimensionnement de ces deux éléments est largement affecté par les déformations du barrage qui ont été analysées moyennant des modèles numériques 2D et 3D.

Ce travail d'analyse a montré que les déformations estimées sont cohérentes avec celles de la littérature technique. Les simulations numériques incluent ainsi des analyses de sensitivité de paramètres clés (comme par exemple la rigidité de la fondation du barrage et la rigidité des matériaux) et ont permis d'identifier des scenarios plus conservatifs pour le projet du système d'étanchéité. En particulier, la conception du système « waterstop » externe envisage l'utilisation de deux matériaux différents avec des fonctions spécifiques :

- *La fonction structurelle est assurée par une couche monodirectionnelle de treillis en fibre de carbone.*
- *La fonction d'étanchéité est assurée par une géomembrane en PVC.*

Les treillis en fibre de carbone à haute rigidité produisent des allongements très petits qui peuvent être supportés par la géomembrane en PVC sans développer des forces de traction significatives qui pourraient affecter la fonction d'imperméabilisation de la membrane. Le joint entre le treillis en fibre de carbone et la membrane en PVC est protégé sur toute sa longueur par une couche constituée par un géotextile en polypropylène plié. Le géotextile est forcé dans l'ouverture du joint par la pression du bassin et, en cas de fuites, assure une protection additionnelle en bouchant le joint et en limitant l'écoulement de l'eau (« collision mat dans la terminologie nautique).

Le système périphérique de joints est conçu pour reprendre les déformations estimées et pour avoir une certaine résilience vis-à-vis de défauts localisés. Le projet de détail du système a été développé au moyen d'une campagne d'investigation spécifique et de prototypes à l'échelle réelle réalisés par Mapei avec l'appui de Studio Pietrangeli.

Keywords

ROCKFILL DAM (BARRAGE EN ENROCHEMENT), STRESS (CONTRAINTE), STRAIN (DEFORMATION RELATIVE), ANALYSIS (CALCUL), WATERSTOP (WATERSTOP), CONSTRUCTION JOINT (JOINT DE CONSTRUCTION), GRAND ETHIOPIAN RENAISSANCE DAM (GERD).