ZAREMA MAY DAY DAM PROJECT IN ETHIOPIA - DAM DESIGN

M. LIGUORI
ELC-Electroconsult, Via 1 Maggio 41, 20021 Milano, Italy

ABSTRACT

The Zarema May Day Dam former Feasibility and Detail Design, by ELC-Electroconsult, determined a 145.00m high embankment dam, with a 730m long crest and reservoir surface of 9,815ha. The study, even if preceded by a Draft Feasibility, was considered, as such, as a basis for future further design developments. An embankment dam has been found to be the best alternative due to the availability of materials and compatibility with the foundation, Client's requirements and the actual workmanship of the already appointed local Contractor. Quality and availability of construction materials have been investigated. The huge quantity of impervious material required by a clay core was not available in proximity to the dam site; materials investigated included asphalt concrete, rock fill, riprap, filters and transition materials. Among the main dam design items special care has been devoted to the asphalt core behaviour modelling, including shear strains distribution in the core and hydro-fracturing check.

1. INTRODUCTION

ELC Electroconsult S.p.A. (Italy), has been commissioned to develop the Feasibility and Detailed Design of the Zarema May Day Dam Irrigation Project in Ethiopia, following the previous Draft Feasibility Design stage. The original previously issued feasibility study was thoroughly reviewed and revised. The main components of the project are: the main dam (Zarema May Day Dam, a 145.00m high embankment, with crest length 730m), the cofferdam (which is part of main dam), one diversion conduit the construction of which was already on-going, a side channel spillway, an intake structure, an approach channel and a Middle Level Outlet, 6.0/8.0 m wide, on the left abutment.

2. DAM TYPE SELECTION

An embankment dam type was found to be the best alternative due to the availability of construction material from quarries and borrow areas and compatibility with the foundation characteristics. Since the huge quantity of impervious material required was not available, the dam has been designed with double type rock fill shells while an asphalt type core (ACC) ensures water-tightness. Dam's zoning design, implying selected free draining and strong materials to form the outer resisting portions of the dam, allowed controlling the dam volume and increasing dam's performance with respect to the original solution. ACC advantages are: relative flexibility against shear since it accommodates larger shear strains with respect to other core materials; favourable deformation conditions with respect to a facing location; self-healing capability of cracks; no erosion under gradients action, possibility of progressive water storage during construction: this met the Owner's requirements. Asphalt concrete has been documented to be virtually impervious when compacted to a void content of less than 3% and its viscoelastic-plastic properties make asphalt core dams especially suited for compressible foundations, possibly due to weathering, where stiffer structures like CFRD and RCC might not be suitable.

3. ASPHALT CONCRETE FEATURES

Zarema Dam design reference has been made to the experiences made in Norway. One reason for this is that asphalt core construction, when compared with its earth core alternative, can still proceed even during rainy weather. By adjusting the bitumen content, the properties of the core can be tailored to local conditions and the core will remain flexible and impervious. The material's ductility and self-healing properties, are clearly demonstrated in practice (for instance in Eberlaste dam). Stress and strain levels in the core, estimated from finite element design analyses, may be used for modelling and checking the field behaviour of the asphalt concrete in the laboratory.

4. DAM SECTION AND CONSTRUCTION MATERIALS

The dam section geometry and zoning has been conceived based on dam site characterization, the possible sources of construction materials and their respective achievable quality and the optimization of the erection and impounding process. Relevant aspects are the foundation rocks features, construction materials types actually identified and their anticipated non homogeneity, a safe approach in the forecast of the free draining character of the materials actually achievable and local Contractor's (already appointed) workmanship. In order to ensure adequate safety factors with respect to the limit stability and excessive deformations and strains, especially those relative to the core, the U/S and D/S slopes have been designed according to 1:2 and 1:1.8 gradients. Materials strength and deformability parameters values have been quantified based on either available tests results or meaningful correlations. Cofferdam's outer slopes are not symmetrical and equal to 2:1 (U/S) and 1.3:1 (D/S). In this way placement of the remaining dam body can be realized independently and the staggered construction process, finalized to the water storage capability during construction, can be optimized. Based on geotechnical investigations results, deep curtain grouting has been envisaged.

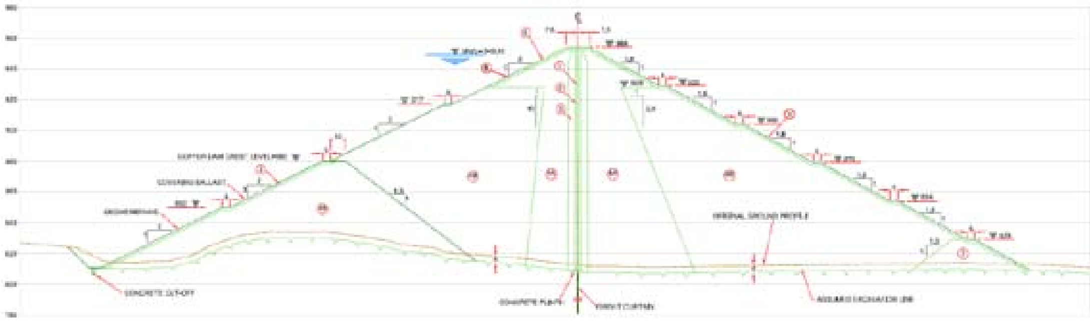

Figure 4-1 : Dam cross section, with indication of Cofferdam

Dam zoning concept is shown in Figure 4-1 The following zones are identified, from inner core to outer shells: Zone 1 - Asphalt Concrete Core; Zone 2 - Filter-Transition; Zone 3 - Transition; Zone 4A - Inner free-draining Rockfill; Zone 4B - Outer free-draining Rockfill; Zone 5 – Slope Protection; Zone 6 – Crown Cap; Zone 7 – Toe Drain. The grain size curves are reported in the Fig. 4-2 diagram in the form of "mean" curves of the respective "fuses", applicable for each Dam body construction material. Suitability of Free Draining Dam materials is to be checked through Unconfined Compressive Strength (UCS) in the saturated state, Water Absorption (WA), ACV (Aggregate Crushing Value) and Los Angeles Abrasion Test. The limit value of permeability coefficient representative of the free draining behaviour, to be checked after compaction operations accomplishment, is ranging between $5x10^{-4} < k < 10^{-3}$ m/s. Other features applicable criteria are: UCS (saturated): > 50 MPa, Water Absorption: < 1% until 1-3%; Los Angeles Abrasion Test: < 25%; ACV < 25.

Asphalt Core bitumen content will normally be between 6.5-7.5% by weight. Bitumen shall be of grade B 80/100. The Specialist Contractor shall report the results of the following initial tests: Penetration, EN 1426, min.70, max.100mm; Ring and Ball, EN 1427, min.43, max. 51 °C; Loss on Heating, (EN 12607-1) max.0.8%. The aggregates shall comply with: Strength according to Los Angeles (LA) < 40 according to EN 1097-2; Flakiness Index < 35 % according to EN 933-3.

Overall representation of the various granular materials constituting the dam body is shown in the following Figure 4-2, representing the average grading curves for each material. The asphalt concrete aggregate size distribution, in terms of fraction passing by weight (%) for each grain size, in mm, is not reported in the diagram and it is summarized hereinafter: 19mm (100), 16mm (90-100), 11.2mm (80-93), 8mm (65-82), 4mm (45-62), 2mm (35-50), 1mm (25-35), 0.5mm (19-28), 0.25mm (15-21), 0.125mm (12-18), 0.063mm (11-15).

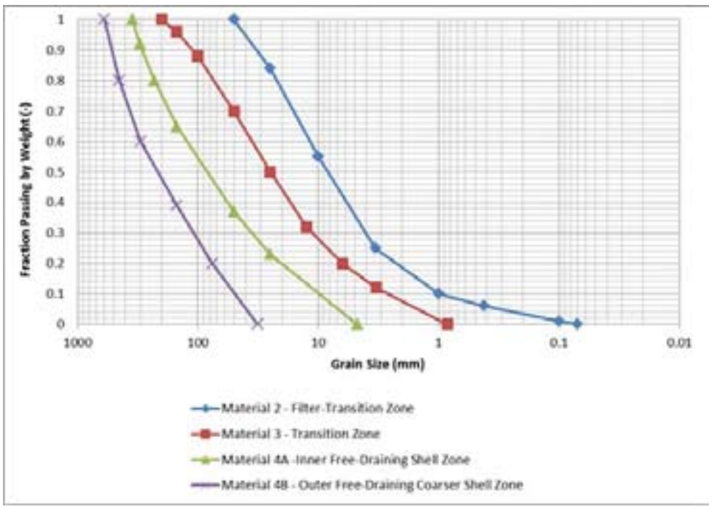

Figure 4-2 : Overall representation of the various granular materials constituting the dam body – average grain size distributions of materials of zones 2, 3, 4A and 4B

5. DAM DESIGN AND VERIFICATION

The behaviour of the dam under static and seismic conditions, during construction, first impoundment and operation of the plant, has been analysed to determine displacements and stress distributions throughout the dam and safety factors. In absence of specific laboratory tests (TX and OED) on large scale samples, the design parameters of the materials, like strength and deformability, have been derived either by applying a suited strength model or from data of similar dam construction materials. Deformational and stability analyses have been carried out through FEM modelling but also by application of the Limit Equilibrium Approach by Slices Method. Deep curtain grouting tip has been set at approx. 80m depth below foundation excavation level. Roc-Science Phase2 version 7.02 was used to carry out FEM modelling, including deformational analysis, stability analysis and seepage analysis; Roc-Science Slide version 6.017 has been used the Slices Method application. The dynamic response of the dam, through Geo-Slope Geo-studio 2004 Ver. 6.02, has also been been studied. Modelling was carried out under plane strain conditions.

6. FEM MODELLING ANALYSIS

The geometry and the mesh of the dam / cofferdam and the staging of construction are shown in the Figure 6-1 and Figure 6-2 below:

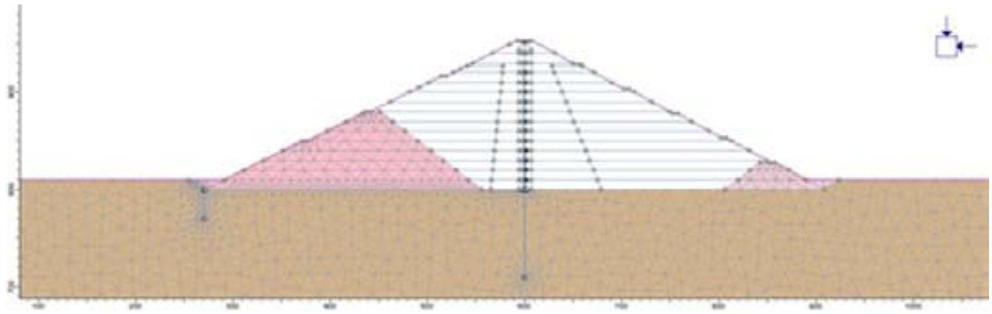

Figure 6-1 : Geometry of FEM model – Cofferdam and Toe Drain - Stage 10

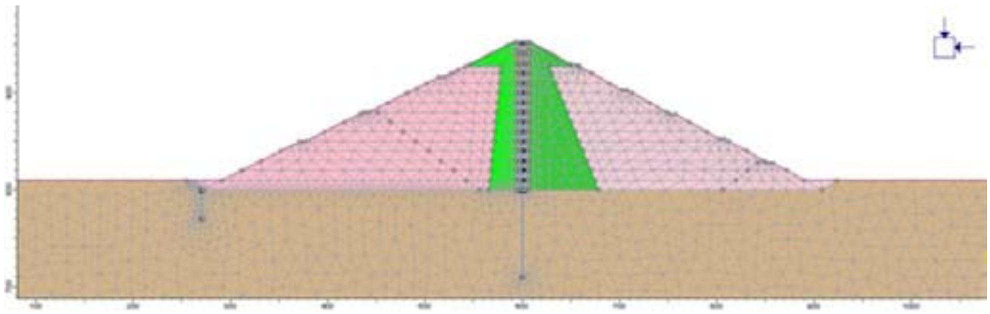

Figure 6-2 : Geometry of FEM model – End of Construction of the Dam - Stage 25

The "Phase2" 2-dimensional elastic-plastic finite element program has been used for calculating stresses and displacements throughout the dam. In order to assess the stability of the dam, the Shear Strength Reduction (SSR) option has been used, to compute a critical Strength Reduction Factor (SRF) for the model, i.e. a Safety Factor estimation. An elastic – perfectly plastic model, with associated plastic flow rule, was implemented (starting from the Mohr-Coulomb failure criterion and plasticity model), complemented by a non-linear elasticity model for the elastic component (Duncan-Chang Hyperbolic constitutive model). Pseudo-static method was adopted to take into account the effects caused by seismic actions on stability. The plasticity model is characterized by the following yield (failure) surface and the plastic potential flow surface:

$$fs = -\frac{I_1}{3}sin(\varphi) + \sqrt{J_2}\left[cos(\vartheta) + \frac{1}{\sqrt{3}}sin(\vartheta)\,sin(\varphi)\right] - c\ cos\,\varphi$$

$$gs = -\frac{I_1}{3}sin(\varphi_{dil}) + \sqrt{J_2}\left[cos(\vartheta) + \frac{1}{\sqrt{3}}sin(\vartheta)\,sin(\varphi)\right] - c\ cos\,\varphi$$

The Duncan-Chang Hyperbolic constitutive model is suited for modelling the non-linear and stress-dependent behaviour of cohesive and cohesionless soils. The input parameters are: Modulus Number (K_e), i.e. a dimensionless parameter representing the Young's modulus; the Unloading Modulus Number (K_{ur}), used to calculate the unloading/reloading conditions modulus; the Modulus Exponent (n), that governs the stress dependency of E_0 on σ_3; the Failure Ratio (R_f), for the shape of the stress-strain curve; the Atmospheric Pressure (P_{atm}); the Poisson's Ratio (v); the Bulk Modulus

Number (K_b), that is a dimensionless parameter characterizing the volumetric change and the Bulk Modulus Exponent (m), that governs the stress dependency of B_t on σ_3. Based on a hyperbolic stress-strain curve and stress-dependent material properties, the following equations are derived for the constitutive model parameters. The tangential modulus (E_t) for a given stress condition, the tangential bulk modulus (B_t) and tangential Poisson's ratio are given by the following equations respectively:

$$E_t = K_E P_{atm}\left(\frac{\sigma_3}{P_{atm}}\right)^n \left[1 - \frac{R_f(1-\sin\varphi)(\sigma_1-\sigma_3)}{2\,c\,\cos\varphi + 2\sigma_3\sin\varphi}\right]^2 E_t = K_E P_{atm}\left(\frac{\sigma_3}{P_{atm}}\right)^n \left[1 - \frac{R_f(1-\sin\varphi)(\sigma_1-\sigma_3)}{2\,c\,\cos\varphi + 2\sigma_3\sin\varphi}\right]^2$$

$$B_t = K_b P_{atm}\left(\frac{\sigma_3}{P_{atm}}\right)^m B_t = K_b P_{atm}\left(\frac{\sigma_3}{P_{atm}}\right)^m \qquad \nu_t = \frac{1}{2}\left(1 - \frac{E_t}{3B_t}\right)\nu_t = \frac{1}{2}\left(1 - \frac{E_t}{3B_t}\right)$$

7. SEEPAGE ANALYSIS AND SLICES METHOD FOR LIMIT EQUILIBRIUM TYPE STABILITY ANALYSIS

A steady-state, saturated-unsaturated finite element groundwater seepage analysis has been carried out by the groundwater analysis Phase2 tool. The hydraulic head and pore pressure distributions and other quantities such as flow rate, hydraulic gradient and discharge velocity are calculated. Slice Method was used to carry out limit equilibrium stability analyses of the dam/cofferdam slopes. In order to filter shallow slip surfaces that, although more critical in principle, are characterized by low confining stresses and higher shear strength than that assumed as average value, a surface filter was applied. For each case under study, three different analyses were carried out: for surfaces having minimum depth of 5m, 15m and 25m. The minimum safety factor, out of those proposed, was retained. Mohr-Coulomb failure criterion was implemented while pseudo-static approach was adopted to take into account the effects on overall stability caused by seismic actions.

8. PERFORMANCE CRITERIA

The performance criteria were selected according to international standards: the selection of load conditions, load cases and minimum safety factors has been made according to USACE EM 1110-2-1902 Slope Stability and USACE ER 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects; also the "load" cases considered in the dam / cofferdam seepage analysis are reported in the following Tables.

Stage	Load Case	Description	Min SF Cat.	Min SF Value	SW	TWL 1/2	CDF	DDF	OBE	MDE	EQ DS/US	EQ DW/ UW
Cofferdam	1	During constr.	III	1.3	x							
	2	During constr. + CDF	IV	1.2	x		x					
	3a	During constr. + OBE	V	1.1	x				x		DS	DW
	3b	" "	V	1.1	x				x		DS	UW
Dam	1	During constr. + TWL1	III	1.3	x	x						
	2	During constr. + TWL2	III	1.3	x	x						
	3	End of constr. + DDF	II	1.4	x			x				
	4a	End of constr. + DDF + MDE	V	1.1	x			x		x	DS	DW
	4b	" "	V	1.1	x			x		x	DS	UW
	4c	" "	V	1.1	x			x		x	US	DW
	4d	" "	V	1.1	x			x		x	US	UW

Table 8-1 : Summary of dam and cofferdam stability analysis load cases

Stage	Load Case	Description	U/S WL (m a.s.l.)	D/S WL (m a.s.l.)	k_1 (m/s)
Cofferdam	4	During constr. + Water at Cofferdam Crest Level	880.0	800.0	N.A.
Dam	5a	End of constr. + Max OL + Mat. 1 Design Perm.	946.4	800.0	$5x10^{-10}$
	5b	End of constr. + Max OL + Mat. 1 Increased Perm.	946.4	800.0	$5x10^{-6}$

Table 8-2 : Summary of dam and cofferdam seepage analysis "load" cases

Stage	**Load Case**	**Description**	**U/S WL (m a.s.l.)**	**D/S WL (m a.s.l.)**	**k1 (m/s)**
Cofferdam	4	During constr. + Water at Cofferdam Crest Level	880.0	800.0	N.A.
Dam	5a	End of constr. + Max OL + Mat. 1 Design Perm.	946.4	800.0	$5x10^{-10}$
	5b	End of constr. + Max OL + Mat. 1 Increased Perm.	946.4	800.0	$5x10^{-6}$

In the Table SW is self-weight, TWL1,2 are Temporary Water Levels, CDF and DDF are flood levels, seismic actions are identified as Operating Basis Earthquake - OBE = 0.08g and Maximum Design Earthquake - MDE = 0.11g. When FEM analyses were applied to the cases shown in the above table, the analyses were carried out under the assumption that the modulus of asphalt concrete is equal to its lower asymptotic value (modulus number K_e = 360, i.e. E = 36 MPa), to account for the effects of viscosity, in accordance with the current engineering practice.

9. STRENGTH, DEFORMATIONAL AND PERMEABILITY PARAMETERS

The rockfill materials have to exhibit adequate strength, limited strength reduction with strain progress and adequate permeability (free draining) in order for any pore pressure build-up, arising with possible cyclic loading, to be controlled. According to the presented zones materials grading curves and other determinable quantities, the evaluation of the relevant main "design parameters" has been carried out; in facts, a detailed analysis of the proper values ranges for the "natural parameters" governing the behaviour and performance of rockfill, is required. The following design parameters, to be implemented for modelling the dam zones behaviour, were examined: Strength, Deformability, Permeability.

Strength – According to Barton [1], as far as the strength, the following form of the peak drained friction angle φ' is envisaged for rockfill: $\varphi' = R \log (S / \sigma'_n) + \varphi_b$, where: σ'_n is the effective normal stress, R is the equivalent roughness and S is the equivalent strength of particles, both derivable from available correlations, φ_b is the basic friction angle. The equivalent roughness R is evaluated as a function of porosity "n" of rockfill after compaction, origin of materials i.e. degree of roundedness and degree of smoothness of particles. The equivalent strength of particles S is evaluated as a function of the uniaxial compressive strength of the rock σ_c and particle size diameter corresponding to 50% passing by weight d_{50}. σ_c has been determined from laboratory and field tests results processing. Design targets in terms of rock pieces strength and porosity of the compacted fill "n" for this project were: σ_c = 72 MPa and n = 25%.

Deformability: the use of large-scale laboratory testing (mainly oedometer and triaxial and plane strain shear testing) to understand the strength and deformation properties of rockfill and obtain the Hyperbolic Model parameters has become important for large projects. The testing confirms the significance of particle breakage on the deformation behaviour of rockfill under increasing applied stresses and on saturation. Increasing modulus was observed with increased compaction effort, decreasing modulus were observed with increasing deviatoric stress levels in triaxial compression tests, in oedometer tests relatively high moduli were observed for compacted rockfill samples up to normal stresses in the order of 800 to 1000 kPa, thereafter the modulus was observed to decrease with increasing normal stress (Marsal 1973). Higher rockfill modulus was observed on un-loading and re-loading at stress levels less than previously experienced, lower modulus was observed for angular more uniformly graded and coarser rockfills, reduced modulus and reduced strength were observed for weaker strength rockfills, besides propensity to collapse on wetting. Considering the data reported by Hunter and Fell [4], the data from laboratory tests campaigns carried out during the preparation of ELC's Projects like *San Roque Multipurpose* and *Sermo Dam Irrigation Project* on similar materials and the recent literature, including that regarding the deformability parameters determinations for various real rockfills and their derivation procedures, it was assumed that the deformational parameters values attributable to the shell zone materials 4A and 4B are as high as reported in the Table 10.1.

Permeability: Since for Zarema May Day Dam a "free draining" rockfill was envisaged, a percentage of particles minus 25 mm less than 40% by weight is due, whereas not more than 30% below 20 mm is to be allowed. However, the effective horizontal permeability may be still too low in such a well-graded rockfill and hence, the control of grain size distribution is essential to govern permeability, i.e. the behaviour of rockfill and dam's performance. According to Kutzner [5], some typical cases concerning ideal and real rockfill grain size distributions and some relevant permeability ratios between respective curves, were considered; in particular, two grain sizes curves close to the Zarema Dam 4A and 4B rockfills could be recognized. From the concept reported in the text, the ratio in terms of permeability between the two materials is even higher than 10. Then, to characterize the 4A shell materials, the following permeability estimation was attempted. The empirical expression by Beyer relates the permeability k of a non-cohesive material (in m/s) to the uniformity coefficient $U = d_{60}/d_{10}$ and the effective diameter d_{10} (in mm): $k = C \times (d_{10})^2$ where $C = 4.5\times10^{-3} \log (500/U)$. In particular, for the 4A, U = 90/5 = 18 and therefore $C = 4.5\times10^{-3} \log (500/18) = 0.006497$ and $k = 0.006497 \times 5^2 = 1.6 \times 10^{-1}$ m/s. Regardless of this theoretical prediction, the criterion for the dam shells zones materials 4A and 4B, in terms of admissible minimum values of permeability coefficient after compaction, to be checked through permeability test, was the following: Material 4A: $k > 5\times10^{-4}$ m/s; Material 4B: $k > 10^{-3}$ m/s.

10. SUMMARY OF THE DESIGN PARAMETERS

Design parameters values for deformational and stability analysis of the dam are summarized in the following Table 10.1. The meaning of the quantities is: Dry unit weight (γ_{dry}) expressed in kN/m^3, Modulus Number (K_e), Unloading Modulus Number (K_{ur}), Modulus Exponent (n), Failure Ratio (R_f), Cohesion (c') expressed in kN/m^2, Friction Angle (φ'_0), Variation of Friction Angle for tenfold variation of normal stress ($\Delta\varphi'$), Poisson's Ratio (v). The asphalt concrete modulus number shown in the table, modulus number $K_e = 360$, i.e. E = 36 MPa, is representative of asphalt concrete modulus in the long-term, accounting for the effects of viscosity and implemented directly in the analysis. In the short term, modulus number of asphalt concrete is higher, and has been assumed equal to $K_e = 9154$, i.e. E = 915.4 MPa. In the present case, the assumption of constant Poisson's Ratio is made in Duncan-Chang Hyperbolic model, therefore parameters K_b and m, as previously defined, are derived parameters. The bedrock is characterized by Mohr-Coulomb failure criterion, where cohesion c = 10.5 MPa, friction angle φ = 35°, Young modulus 20 GPa (bedrock acts as a high strength and rigid foundation, when compared to the dam materials). The permeability coefficients for seepage analysis of the dam are provided as well. Further cases analysed cover the possibility for the unforeseen asphaltic core loss of water-tightness, simulated through an average permeability increase up to $k = 5.0 \times 10^{-6}$ m/s, corresponding to an air void content increase up to 10 %.

Table 10-1 : Geotechnical characterization of the materials of the different zones for dam / cofferdam stability analysis

Zone	Description	γ_{dry}	K_e	K_{ur}	n	R_f	c'	φ'_0	$\Delta\varphi'$	n
1	Asphalt concrete core	24	360	360	0.00	0.70	800	35	0	0.45
2	Filter- transition zone	21	900	1350	0.40	0.70	0	40	4	0.30
3	Transition zone	21	1000	1500	0.42	0.70	0	40	3	0.30
4A	Inner free-draining (rockfill) shell	22	1100	1650	0.49	0.73	0	42	5	0.30
4B	Outer free-draining (rockfill) coarser shell	22	1200	1800	0.50	0.73	0	45	5	0.30

The permeability coefficients, in m/s, of the materials of the different zones and foundation for dam seepage analysis are as follows: Asphalt concrete core: 5.0×10^{-10}, Filter-transition zone: 1.0×10^{-3}, Transition zone: 2.5×10^{-3}, Inner free-drain. (rockfill) shell: 5.0×10^{-3}, Outer free-drain. (rockfill) coarser shell: 5.0×10^{-2}, Foundation rock (first 4m thick layer from El. 796m to El. 800m): 1.0×10^{-6}, Foundation rock (below El. 796m): 5.0×10^{-7}, Curtain grouting: 2.0×10^{-7}, Concrete: 1.5×10^{-11}.

11. RESULTS OF SEEPAGE ANALYSES

In the following Fig. 11-1 and 11-2, the results of the seepage analyses, in graphical terms, in terms of Total Head contour lines [m] with flow lines and phreatic line within dam body, for the Load Case 5a – Seepage Max OL and k1 = 5e-10 m/s and the Load Case 5b – Seepage Max OL and k1 = 5e-6 m/s, as a malfunctioning condition of the core, inducing significant rise of the phreatic line, are shown.

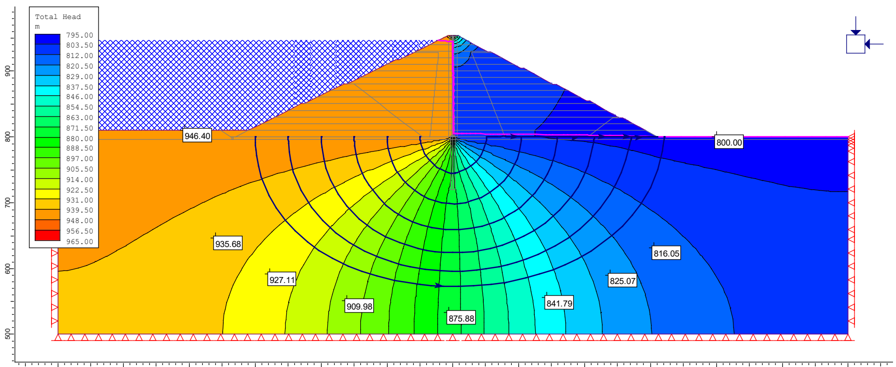

Fig. 11-1 : Load Case 5a – Seepage Max OL and k1 = 5e-10 m/s - Total Head contour lines [m] with flow lines and phreatic line within dam body

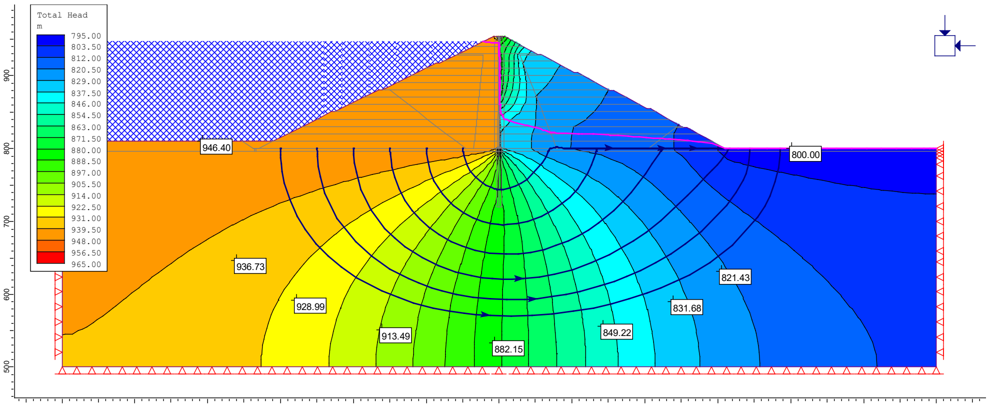

Fig. 11-2 : Load Case 5b – Seepage Max OL and k1 = 5e-6 m/s - Total Head contour lines [m] with flow lines and phreatic line within dam body

12. DEFORMATIONAL, STABILITY ANALYSIS AND MAXIMUM SHEAR STRAIN IN THE CORE RESULTS

Construction staging was implemented in the FEM model for deformational and stability analysis. Lifts of 10m each were modelled. According to dam construction and reservoir impounding, the stages were defined along with the associated water levels, attained during the progressive impounding. At each stage, the displacements and stresses throughout the dam/cofferdam were determined and the most critical stages exported from FEM to be analysed by Slices Method. The final stage, occurring after the end of construction and the stabilization of the water flow through and underneath the dam, was implemented in the FEM model for seepage analysis (Fig. 11-1/11-2). Shear strains in the dam core have been analysed to investigate the actual deformation process, under the assumption that the modulus of asphalt concrete varies in time ("short term" with modulus E = 915.4 MPa while in the long-term the modulus drops to asymptotic value E = 36 MPa). Since the values of shear strain evaluated for seismic conditions simulated by pseudo-static approach are considered not meaningful (see (*) in Table 12.1), the first contribution to the core maximum shear strains considered is that due to the load conditions: SW (Self Weight) and DDF (Dam Design Flood). The most significant results of the analyses done are reported: the effects of earthquake on core shear strains (second contribution) have been investigated through a dynamic response analysis, commented in a following section.

Table 12-1 - Summary table of the most significant results of deformational and stability analysis of dam and cofferdam

Stage	Load Case	Description	Min SF Value	FEM		Slices Method		
				Mat. 1 Max Shear Strain [-]	SRF	Min 5m depth SF	Min 15m depth SF	Min 25m depth SF
Cofferdam	3b	During construction + OBE	1.1	-	1.30	1.11	1.15	1.23
	3	End of construction + DDF	1.4	0.015	2.10	1.67	1.99	2.03
Dam	4d	End of construction + DDF + MDE	1.1	(*)	1.50	1.17	1.33	1.35

Impounding produces a reduction in effective stresses and the consequent heave of the upstream shell. U/S side of the core is loaded by the upward movement of the shell and horizontal water pressure. Vertical stress in the core is reduced due to the buoyancy effect acting on the shell and also by the arching effect within the core due to the varying core modulus from short to the long-term. Principal strains ε_1 and ε_3 are considered. Maximum strains admissible to save imperviousness have been given by Breth & Schwab in terms of 14% for maximum axial and 6% for transversal component. The shear strains field is given by the quantity $\gamma = 2*\varepsilon_s = \varepsilon_1 - \varepsilon_3$. In particular for the Load Case 3, the effect of the application of the final Asphalt Core modulus leads to $\gamma = 0.015$ while, considering the effects on γ due to variation of core modulus, the maximum resulting shear strain was found to be still well smaller than the mentioned limiting value (2*0.024 < 0.06), as described by the diagram reported in Fig. 12.1 (ε_s).

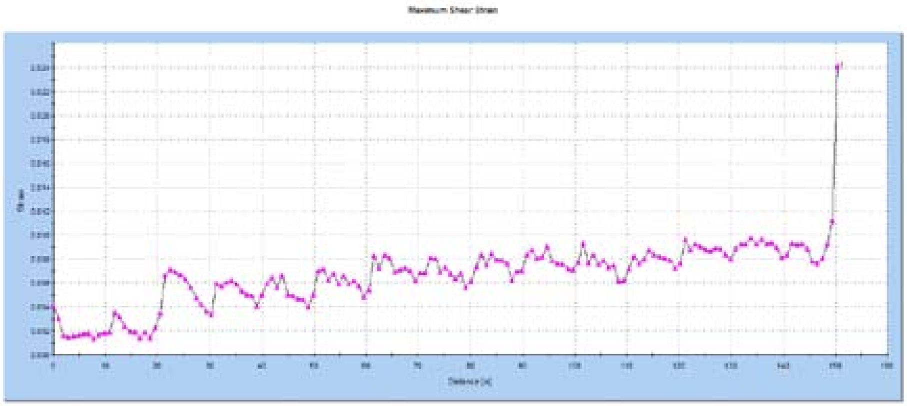

Figure 12-1 : Dam – Load Case 3b – Long term + DDF with varying modulus- Stage 27 – Maximum Core Shear Strain ε_s [-] – Plot along core from crest to base

13. VERIFICATION OF CORE SEISMIC SHEAR STRAINS CONTRIBUTION FROM DAM'S DYNAMIC RESPONSE

The dynamic response of the dam, (Geo-Slope Geostudio 2004 Ver. 6.02), due to a proper seismic ground motion time history, representative of the site MDE, has been investigated. The analysis is based on accelerograms from international database and corrected to be project-specific. FEM modelling has been used to determine the displacements and stress-strain distribution throughout the dam under earthquake, with particular reference to shear strains within the asphaltic core. The peak horizontal accelerations at dam crest are in the range of 0.2-0.3g. The peak horizontal displacements at dam crest are in the range of 15 to 25 mm, which are deemed compatible with requirements on dam stability and deformability. Figure 12-1shows the values of maximum XY shear strain within the dam core, for selected nodes and selected time steps, vs. the elevation of the nodes. The maximum value was retained as maximum shear strain. From superposition principle, it is concluded that the sum of maximum shear strain calculated for static conditions $\gamma_{xy} = 5 \times 10^{-2}$ and maximum shear strain $\gamma_{xy} = 1 \times 10^{-4}$ calculated for dynamic conditions, is still admissible being < 0.06.

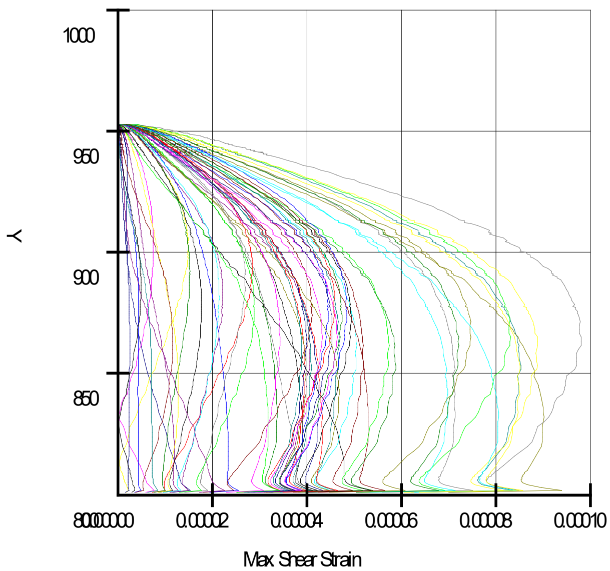

Table 13-1 : Dynamic Analysis– Max Shear Strain, "All" time steps

14. HYDRAULIC FRACTURING POTENTIAL IN THE ASPHALT CORE

For this verification the load condition of interest is: SW Self Weight and DDF Dam Design Flood. Hydraulic fracturing potential is analysed comparing the minor total principal stress σ_3, acting in the dam core elements, with the applied hydraulic pressure. According to the recent experience, gained in connection with the failure of Teton dam, hydraulic fracturing does not occur until (acceptability criterion to be guaranteed for the hydraulic fracturing verification): u = Pwp $\leq p_{hf} = 1.35\ \sigma_3 + \sigma_t$ where u is the pore water pressure, σ_3 is the minor principal total stress and σ_t is the tensile strength of asphalt concrete. In order to calculate σ_t by Mohr- Coulomb criterion construction for the core material, it is recalled that $\varphi = 35°$, c = 0,8 MPa and $\sigma_t = (2*c) / \tan(45°+ \varphi/2) = 0{,}83$ MPa and then it is assumed $\sigma_t = c$. From the diagram reported Figure 14-1, even if it is assumed that $\sigma_t = 0{,}8$ MPa, derived from the linear interpretation of the

Mohr-Coulomb criterion, based on the cohesion and shear resistance angle assumed from literature, on the safe side, it can be concluded that the core is sufficiently safe against hydraulic fracturing provided that the widened (1,20 m x 2= 2,40 m) stretch of the asphalt core at the base contact with the concrete plinth is kept constant at least for an height in the order of 1 m.

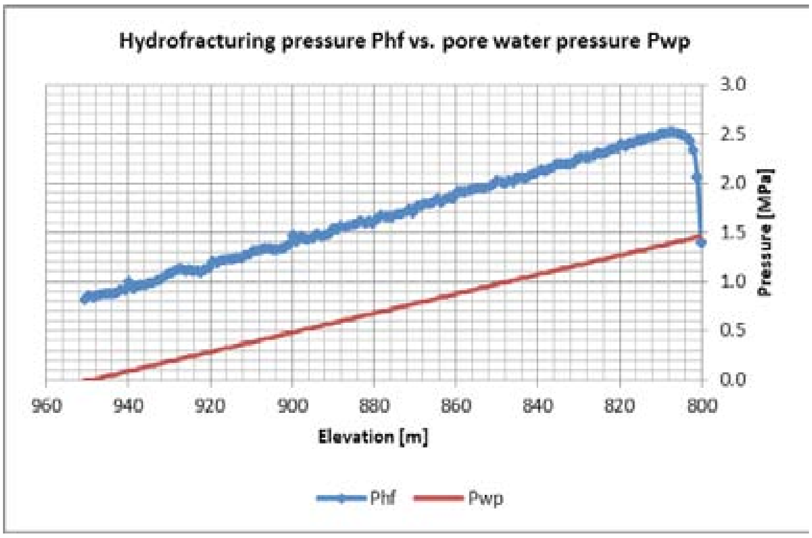

Figure 14-1 : Dam – Load Case 3b – End of construction (long term) + DDF - Stage 27 – Hydrofracturing. Pressure vs. PWP [MPa]

15. CONCLUSIONS

Zarema Dam's design concept has been based on several experiences made in Norway. Asphalt core construction, when compared with earth core alternative, can proceed even during rainy weather and allows progressive water storage during construction, meeting Owner's requirements in this case. The asphalt concrete core adjusts to the deformations in the embankment and to differential settlements in the dam foundation. The material's ductility and self-healing properties are clearly demonstrated in practice. Stress and strain levels in the core, obtained from finite element design analyses, are acceptable and it can be concluded that the core is safe against shear failure and hydraulic fracturing provided that the widened (2,40 m) stretch at the contact with the plinth is kept constant at least for a 1 m height.

REFERENCES

[1] Barton, N. and Kjaernsli, B., *Shear strength of rockfill*, ASCE, Vol. 107, No. GT7, July, 1981

[2] Duncan, J. M. and Chang, C. Y., *Nonlinear analysis of stress and strain in soils*, Journal of the Soil Mechanics and Foundations Division, ASCE, 96(SM5), 1629-1653, 1970.

[3] Høeg, K., Asphalt-concrete core embankment dams, 1997

[4] Hunter, G. and Fell, R., *The deformation behaviour of rockfill*, UNICIV Report No. R-405, The University of New South Wales, Sydney, Australia, January 2002

[5] Kutzner, C., Earth and rockfill dams, Principles of design and construction, A.A. Balkema, Rotterdam, Brookfield, 1997