

Seepage Control Measures for the Papadia Dam Foundation: Design, Construction and Performance

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ABSTRACT: In order to secure the watertightness of the foundation of the 67m high Papadia rockfill dam, a 35.50m maximum depth cut-off wall was constructed in the Pleio-Pleistocene fluvio-glacial deposits and an over 70m deep triple row grout curtain in the gneissic bedrock underneath and on the same axis in the river bed. The cut-off wall consisted of a one row of plastic concrete secant piles. A rigorous Quality Assurance program was implemented during construction of the cut-off wall, and a great number of tests were performed, in the field as well as in the laboratory, in order to validate the integrity of the structure. Two years after reservoir filling, monitoring data confirm the satisfactory performance of the seepage control measures applied.

INTRODUCTION

Papadia Dam, is located on the Geropotamos River, Florina Prefecture, Greece. The main purpose of the dam is to provide cooling water for the 330MW Public Power Corporation of Greece S.A. (PPC S.A) Meliti Thermal Power Plant, situated 15km away. In addition, water is provided for irrigation and to cover the domestic needs of the nearby villages, while 4GWh is also produced annually.

Construction of the project started in year 2001 (Contractor Odon-Odostromaton S.A. initially and then ALTE S.A). After a two years interruption (2004 – 2006), the project was completed in 2008 (Contractor Actor S.A. – Pan-techniki S.A).

GEOLOGY

The geology of the Papadia reservoir area consists of metamorphic rocks of the Palaeozoic era (granitic gneisses – schists). The rocks are weathered at a limited depth and covered by a thin layer of debris. A particular feature of the geology of the area is a zone of Pleio-Pleistocene deposits (PLS) of fluvio-glacial origin, filling an old river channel in the bedrock. The zone is oblique to the river valley, crossing it from the right at the upstream to the left at the downstream (Figure 1).

The PLS are dense, well graded silty sandy gravels, slightly preloaded due to the prior glacial action. A limited number of clay lenses, as well as quartz or granite cobbles and boulders of significant size (up to 10m³ in one case, but usually up to 1m³) are encountered in the PLS.

Water tests in the PLS during the design stage provided coefficient of permeability values as high as 10⁻⁵m/sec. Hence, securing the water-tightness of this formation was deemed of primary importance. Artesian pressures up to 100KPa were also encountered in the bedrock, usually at the contact with the PLS.

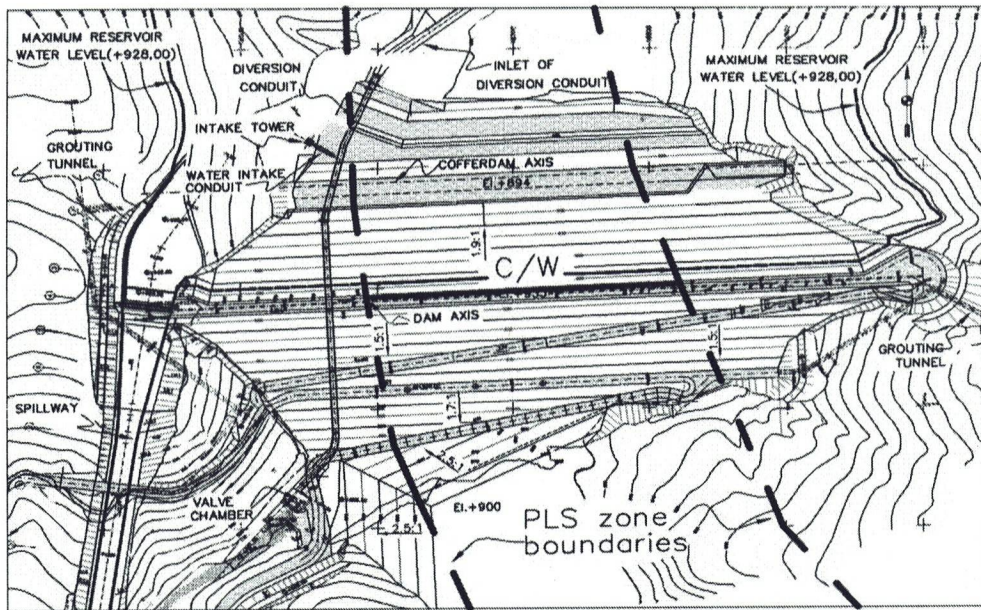


FIG. 1. Papadia dam layout.

SEEPAGE CONTROL MEASURES

The dam axis was placed on purpose where the PLS zone, crossing the valley, coincides in plan with the river bed (Figure 1). This was decided in order to limit the depth of intervention in this zone. The maximum depth of the PLS there is of the order of 35m from the original ground surface.

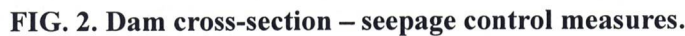
The measures foreseen to secure the foundation water-tightness consist of:

- construction of a grout curtain at the dam axis and into the bedrock (Figure 2);
- construction, on the same axis and into the PLS, of a cut-off wall, consisting of intersecting plastic concrete piles (Figure 2);
- drilling of relief wells under the downstream dam shell into the PLS, penetrating 15m into the underlying bedrock.

Grout Curtain

Layout

The grout curtain, reaching a depth of 70m in the river bed, consisted of 3 parallel rows of vertical holes drilled at 3m distances along the dam axis. Drilling of the holes was performed from the core foundation level. The curtain penetrates the rock after drilling up to 35m into the PLS. The upstream row of holes was drilled first, followed by the downstream and then by the central row. In each row, primary holes were drilled first at 12m distances using rotary drilling rigs, followed by secondary holes at mid-distances, then by tertiary etc, the latter with down-the-hole equipment. Holes of higher order were drilled when cement absorptions of adjacent lower order holes at any particular stage exceeded 50kg/m.



Evaluation of Grout Curtain

“Virgin” rockmass permeability (i.e. of the untreated rock) was determined by evaluating Lugeon tests in primary holes of the upstream row, while “residual” permeability (i.e. of the treated rock, after grouting) was evaluated by Lugeon tests in the central row. Contours of equal coefficients k (in cm/sec) are presented in [Figure 4](#), for “virgin,” as well as “residual” rock permeability. The decrease in permeability of the bedrock after the grouting process is apparent from this figure.

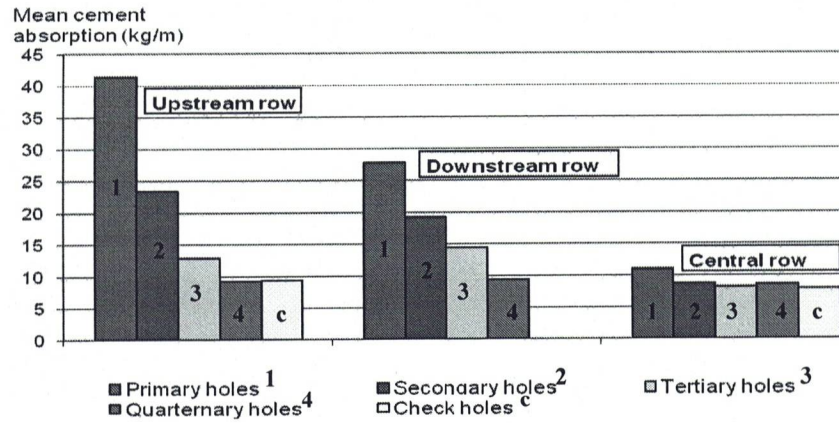


FIG. 3. Mean cement absorption of grout curtain in the river bed.

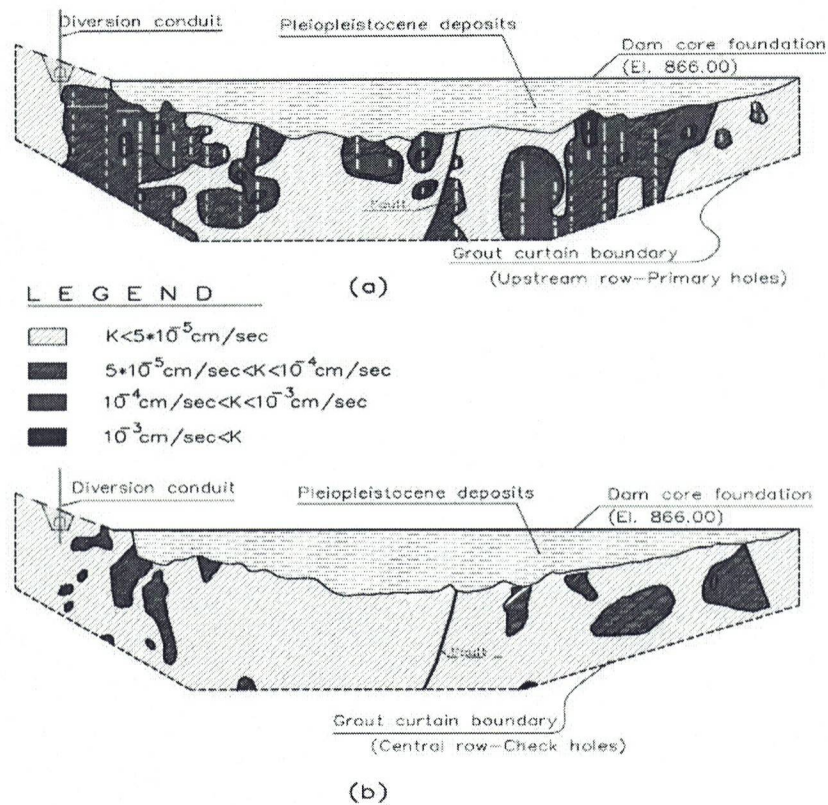


FIG. 4: Bedrock permeability (looking upstream) (a) upstream row primary holes, (b) central row check holes

Cut-off Wall

Layout

The cut off wall (C/W) consisted of a single row of secant piles of plastic concrete. A 4m embedment (tip) of the C/W in the core material was specified. The core material was placed up to 5% wet of the optimum from the core foundation level and up to 5m above the C/W tip, in order to ensure plastic behaviour, alleviating in this way undesirable stress concentrations in the area of the pile tip.

After completion of the grout curtain in the underlying bedrock, 279 piles of 7,223m total length were constructed, working from a platform on the clay core, 5m above foundation level. The piles were of 1.27m diameter and drilled at 0.83m centres. This created a theoretical chord length of 1m at two adjacent piles, and of at least 0.70m at 30m depth, if assuming the specified 0.50% maximum deviation of the holes to have occurred at opposite directions.

The final length of the C/W was 230m with a total area of 6,000m². A longitudinal section of the C/W, with the check holes positions, is shown in [Figure 5](#).

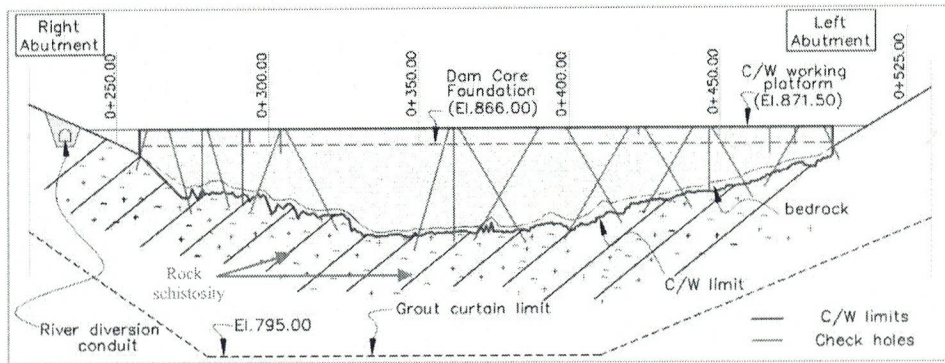


FIG. 5. C/W longitudinal section – check holes

Plastic Concrete Laboratory Tests

A great number of laboratory tests, varying proportions of plastic concrete ingredients within the range used in other projects worldwide, was carried out before construction of the C/W. The final mix adopted is presented in [Table 1](#).

Table 1. Plastic concrete composition.

Material	Quantity/m ³
Cement (kg)	150
Dry Bentonite (kg)	35
Water for Bentonite Hydration (lt)	350
Free Water (lt)	85
Sand (kg)	675
Gravel (kg)	675

The mix was tested by numerous tests that were conducted on cylindrical samples. The density (20.0kN/m^3 wet and 18.0kN/m^3 dry), moisture content (around 20%), as well as the 28 days UCS (1.00 to 1.30MPa) and secant deformation modulus at 5% strain at a 200KPa cell pressure ($<200\text{MPa}$, stress-hardening behaviour) tests provided consistent and acceptable results. Permeability tests, conducted at gradients from 50 to 300:1, gave permeability coefficient values less than 10^{-8}m/s , while pinhole tests indicated that the plastic concrete (even without coarse aggregate, for testing purposes) was very much in the “not susceptible to erosion” category. This provided a high level of confidence for the long-term performance of the wall.

Cut-Off Wall Construction

The C/W construction was conducted by a specialty subcontractor (Edrasis – C. Psallidas S.A.) working under very detailed technical specifications and observing a well-defined Method of Statement and Quality Plan. The total duration of the C/W construction, completed in two different periods, was approximately 10 months.

One or two pile construction groups were used for the C/W construction, i.e. a Casagrande C600 H40 and (occasionally) a Casagrande B250. The casing was driven by Casagrande GC72/1250 and GCL1500/ACE hydraulic oscillators, always advancing the drilling bit by at least 0.50m. This helped to avoid problems that might arise if loose rock blocks or boulders were encountered, something to be anticipated in the PLS formation.

Piles were drilled consecutively, primary ones first, followed by construction of the secondary. These were drilled at mid-distances between the primaries, 5 days at least after pouring the latter, so as the plastic concrete to have achieved a UCS higher than 0.50MPa. Only two piles encountered large boulders ($>1\text{m}$ diameter), leading to drilling difficulties. These piles were declared as “problem piles”, the casing was withdrawn, the pile was temporarily backfilled and a new drilling tool was mobilised (bull nosed, diamond impregnated bit), to allow continuation of penetration to full depth.

After dewatering the pile hole up to the bottom, pouring of the concrete was quick and continuous, generally lasting 2-3 hours. The casing tip was smoothly retrieved during pouring, but was held at least 3m below the plastic concrete level at any given time. Records indicate that the minimum specified penetration depth (1.50m) into the hard rock under the PLS was relatively easy to achieve with the machinery available, typically taking 1 to 2 hours to drill, whereas the average industrial productivity for a 10 to 12-hour day was around 35m (one deep pile or 2 shallower ones). Only one major production delay (5 days) due to mechanical breakdown was recorded.

In some projects where trench cutters for panel construction were used in similar to Papadia foundation conditions, filling the trench excavation with a cement bentonite mixture was found not adequate for wall stabilisation. There, additional measures (pressure grouting) had to be applied in advance of trench excavation, as described by Balian (2007). That was avoided with the construction method adopted in Papadia, significantly reducing costs and time delays.

Field QA/QC

For each pile, the station, the deviation from vertical, the date of construction, the total depth, the secant (chord) overlap, the volume of concrete placed and data relating to rate of gain of concrete UCS was recorded.

After completion of drilling of each pile, verticality checks were performed, using a laser-beam apparatus developed by the site personnel. Results were very satisfactory, in all but 5 cases deviation from the vertical being less than the maximum specified. The piles with larger deviations were replaced by drilling new ones.

For each batch of concrete used, slump, density and temperature were measured at the batching plant and slump was again measured at the pile location. For every pile, six cylindrical specimens (200x100mm), as well as at least 9 cubic ones (100x100x100mm) were taken for testing the rate of gain of UCS, at 7, 28 and 56 days after pouring. Tests were performed at the site laboratory and results (Table 2) were consistent with those obtained during the mix acceptance tests.

Table 2. Plastic concrete UCS (site laboratory records, in MPa).

Age	Average	Minimum	Maximum
7	0.75	0.33	1.39
14	0.90	0.40	1.60
28	1.08	0.50	1.70
56	1.27	0.83	1.93

C/W in Situ Tests

A significant number of in situ tests (water permeability, SPT and CPT) were conducted in check holes drilled with full core recovery in the completed C/W, as described in Bruce & Milligan (2006). A total of 18 check holes (Figure 5) were drilled, in piles ranging in age from 20 days to over 2 years. Coring was continued into the basal bedrock, in order to permit the concrete-rock contacts to be observed. Summary observations are as follows:

- Cores of plastic concrete showed remarkable homogeneity throughout, with no evidence of very soft/unhardened zones.
- Contacts between piles and rock in adjacent piles of different phases (i.e., primary-secondary etc.) were excellent. Indeed, inter-pile contacts were in fact impossible to judge visually in the samples.
- Wet density varied from 1.89 to 1.95KN/m³
- Maag test results in the piles were uniform, averaging a permeability coefficient around $5 \cdot 10^{-7}$ m/s. Due to leakage around the casing, it is reasonable to believe that these data may be an overestimation of the actual in situ conditions (to be noted that the results calculated from the two water recovery tests gave values of $5 \cdot 10^{-8}$ and $5 \cdot 10^{-9}$ m/s respectively).
- The relatively low strengths achieved from cores (typically 30 to 50% of values recorded from cylindrical samples from batch) is considered to be a result of the disturbance caused by the coring process. This “reduction factor” is commonly found when comparing similar sources of samples in Deep Mixing and jet grouting projects.

- Sieve analyses on disaggregated core samples from 22 samples shows that no in situ backfill segregation had taken place.

Mass concrete samples were taken from the upper 1.50m in certain piles and cubes were saw-cut from these samples for UCS testing. These yielded samples somewhat less disturbed than the cored samples. All saw-cut samples (as well as cored samples) were wrapped in cloth and covered with wax immediately upon retrieval. This process greatly enhanced the validity of the subsequent testing results. The results from these samples with respect to wet density, strength (UCS values from 0.63 to 1.38MPa) and rate of gain of strength were closer to data from samples taken during batching.

From the SPT and CPT testing, no significant anomalies (e.g., soft, unhardened zones etc) were encountered in the C/W.

Long Term Laboratory Tests

In order to check evolution of the plastic concrete properties in the long term, a series of tests (UCS and triaxial) were conducted in PPC S.A Central Laboratory. A great number of samples were prepared and were preserved under different conditions (in wax, in wet chamber and in water), in order to simulate as far as possible in situ conditions.

Tests on samples were carried at 7, 28, 365 and 730 days. Results confirmed that in general the plastic concrete gained strength in the long term ([Figure 6](#)). The differences in early UCS strengths recorded in these tests, compared to the values of Table 2, are attributed to late strength development of the particular cement batch used.

PUMPING TESTS

The efficiency of the seepage control measures was checked after completion of the structure, by a series of pumping tests. Pumping was carried out in different stages, both from upstream and downstream of the C/W, recording discharge as well as water levels in piezometers in the foundation (vibrating wire and stand-pipes).

Evaluation of the data collected from the tests verified the satisfactory performance of the seepage control measures (and in particular of the C/W, as described by Bruce and Milligan (2006) and Oikonomidis et al. (2010), allowing finally the closure of the diversion channel.

CONCLUSIONS

Two years after Papadia reservoir filling, leakages are less than 300 l/min. This value is quite low, as it corresponds to less than 1% of the average annual river discharge. Water levels in all foundation piezometers have stabilised.

The water table level in the foundation downstream of the C/W practically coincides with the downstream drainage blanket elevation as mentioned in Anastopoulos et al. (2010a and 2010b). No indications of erosion or wash-out of fines from the foundation is observed.

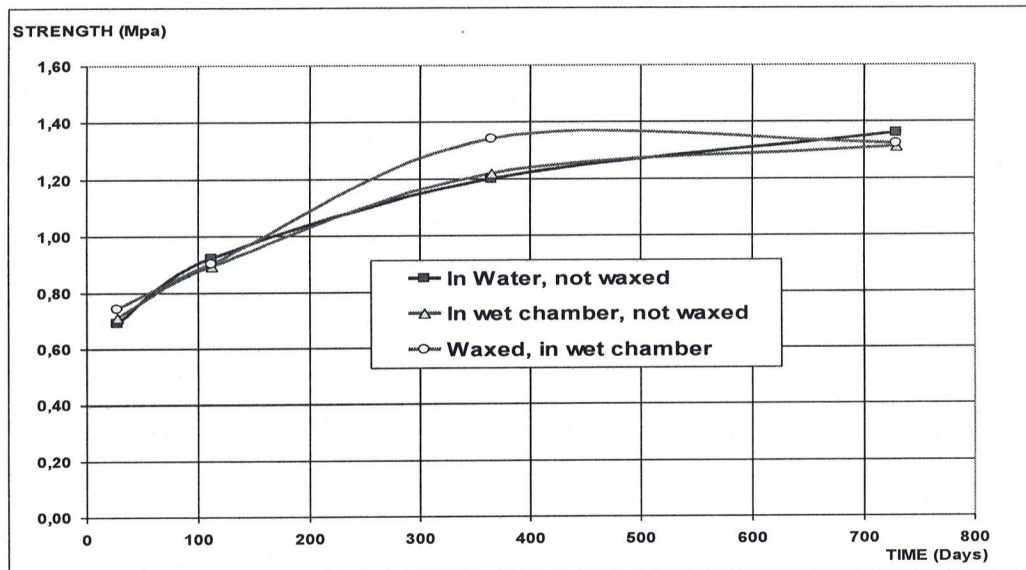


FIG. 6. Long term evolution of plastic concrete USC (mean values).

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