Construction of Papadia secant pile plastic concrete cut-off wall

Construction d'un diaphragme de pilotis intersectés en béton plastique à Papadia

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ABSTRACT

In order to secure the water-tightness of the foundation of the 67m high Papadia rockfill dam, a cut off wall up to 35.50m deep, formed by secant piles of plastic concrete was constructed in the Pleio-Pleistocene fluvio-glacial deposits at the river bed. A rigorous Quality Assurance Program was implemented during construction. A great number of tests, both in the field as well as in the laboratory were performed in order to validate the quality of the structure.
Two years after filling the reservoir, monitoring data confirm the satisfactory performance of the seepage control measures

RÉSUMÉ

Afin d'assurer l'imperméabilité de la fondation du 67m haut barrage en remblai de Papadia, un diaphragme, d'une profondeur allant jusqu' a 35.50m, formé par des pilotis intersectés en béton plastique, a été construit dans les couches fluvio-glaciaires du plio pléistocène au fond de la vallée. Un programme rigoureux de contrôle de qualité a été applique pendant la construction. Un grande nombre d'essais sur place, tandis qu'en laboratoire, ont été réalises pour assurer l'intégrité de l'œuvre.
Deux ans après le remplissage du réservoir, l'évaluation des résultats des instruments indique la performance satisfaisante des mesures appliqués.

Keywords: cut-off, diaphragm wall, plastic concrete, secant pile

1 INTRODUCTION

Papadia Dam, on the Geropotamos River, Florina Perfecture, Greece, was completed in 2008. The main purpose of the dam is to provide cooling water for the Public Power Corporation of Greece S.A. (PPC S.A) 330MW Meliti Thermal Power Plant, situated 15km away. In addition, water for irrigation and domestic needs is provided to the villages nearby, and 4GWh of energy is annually produced.

Construction started in 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. – Pantechniki S.A).

2 GEOLOGICAL CONDITIONS

The geology of the Papadia reservoir area consists of metamorphic rocks of the Palaeozoic era (granitic gneisses and schists). The rocks are

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weathered to 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).

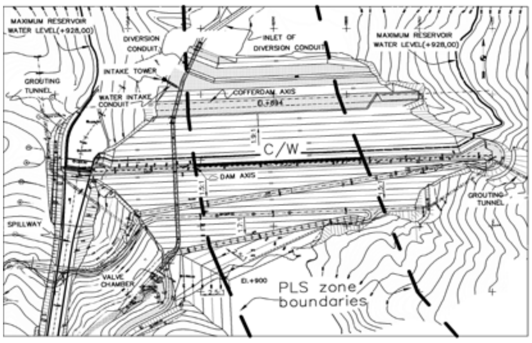

Figure 1. Papadia dam – Layout

The PLS sediments 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 (in one case of a volume of up to 10m^3, but usually up to 1m^3) are encountered in the PLS.

Water tests in the PLS during the design stage provided coefficient of permeability values as high as 10^{-5}m/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, at the contact with the PLS.

3 SEALING MEASURES

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

The measures that were foreseen in order to secure the foundation water-tightness were:

- construction of a grout curtain at the dam axis, 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 (Figures 2, 3 and 4)
- drilling of relief wells under the downstream dam shell into the PLS, penetrating 15m into the underlying bedrock (Figure 2).

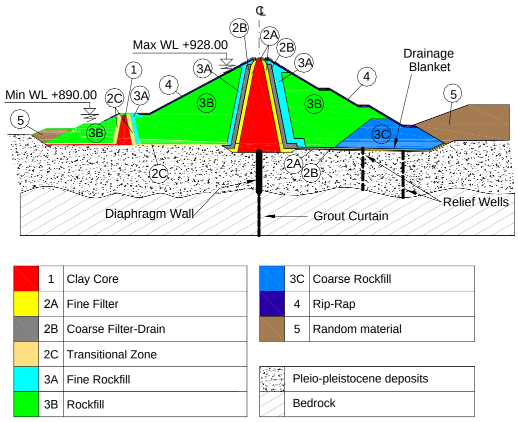

Figure 2. Dam cross-section – Seepage control measures

4 CUT-OFF WALL

4.1 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 the plastic behaviour of the core, alleviating in this way undesirable stress concentrations in the area of the tip.

After completion of the grout curtain in the underlying bedrock, 279 piles of 7,223m total length were constructed from a working platform on the clay core, 5m above the core foundation.

The piles were of 1.27m diameter and were drilled at 0.83m centres (Figure 3). This created a theoretical chord length of two adjacent piles of 1m, and at least 0.70m at 30m depth assuming the specified 0.50% maximum deviation of the holes to have occurred at opposite directions.

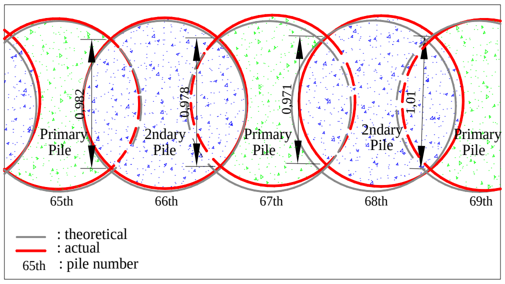

Figure 3. C/W - actual cord lengths at 30m depth (detail)

The final length of the C/W structure was 230m, the total C/W area 6,000m^2 and the total volume of plastic concrete poured 2,522m^3. A longitudinal section of the C/W, with the check holes positions, is shown in Figure 4.

4.2 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	150kg
Dry Bentonite	35kg
Water for bentonite hydration	350lt
Free Water	85lt
Sand	675kg
Gravel	675kg

Bentonite used for the acceptance tests and used during the first C/W construction period (2003) was from Milos Island (LL = 649%). During the second period (2005 – 2006) the bentonite came from Bulgaria (LL = 711%). That involved minor changes in the bentonite / water ratio, from 1:10 to 1:11. The gradation of the coarse aggregate consistently showed 99% less than 13mm and 70% less than 10mm, placing it at the fine extreme of the acceptable range.

An extensive array of tests was conducted on cylindrical samples. These included determination of:

- moisture content
- dry and bulk density
- permeability
- strength (UCS, UUPP, CUPP, CD)
- elastic modulus
- erosion resistance (pinhole test / ASTM D4647-93)

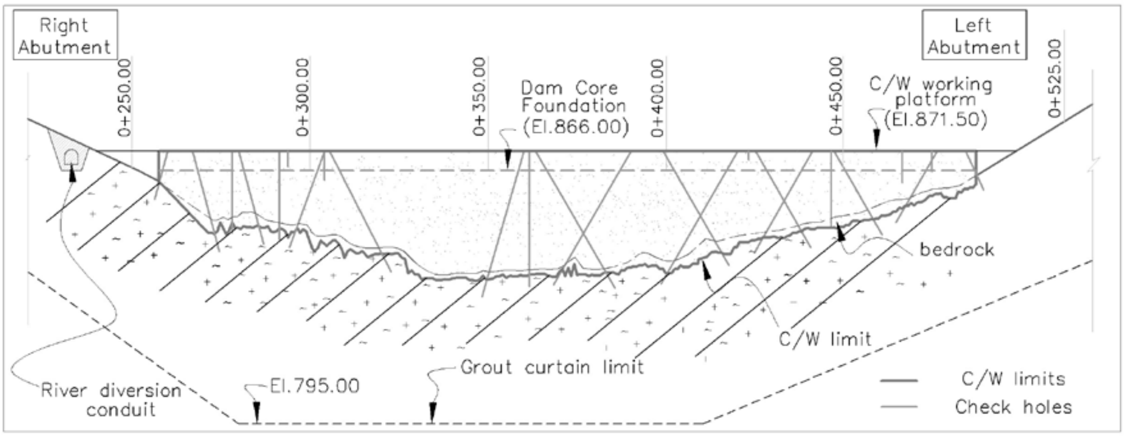

Figure 4. C/W longitudinal section – check holes

The density (20.0kN/m^3 wet and 18.0kN/m^3 dry) and 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 (<200MPa, 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.

4.3 Cut-Off Wall construction

The C/W construction was conducted by the 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.

Depending on the site and schedule conditions, one or two pile construction groups were used, i.e. a Casagrande C600 H40 (max. torque =360kNm, vertical force =400kN) and (occasionally) a Casagrande B250 (max. torque =220kNm, vertical force =240kN). As for the ancillary equipment, a 41t Liebherr LH 841 HD and a 40t Senebogen 640 crane were also used. All drilling was carried out using steel casing, Casagrande type (l =3,00 – 5,00m, Dext =1250mm, Dint =1150mm). The casing was driven by Casagrande GC72/1250 and GCL1500/ACE hydraulic oscillators (max.torque =2,340kNm, vertical force = 2,000 – 3,000kN). The casing always advanced 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. It has to be mentioned that in some diaphragm wall projects in similar environments where trench cutters were used, filling the excavation with a cement bentonite mixture was not found adequate for the trench wall stabilisation. In these cases additional measures (pressure grouting) had to be applied in advance of the C/W excavation [1]. That was not necessary for the Papadia C/W, significantly reducing costs and time delays.

Piles were drilled consecutively, primaries first, these being the piles drilled totally within the PLS. Construction of secondary piles (partially intersecting the primaries) at mid-distances followed, 5 days at least after pouring primary ones, when the plastic concrete had developed a UCS higher than 0.50MPa.

Only two piles encountered large boulders (>1m diameter, granitic gneiss), 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 into the hard rock underlying the PLS (1.50m) 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.

4.4 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 were recorded. The ratio of concrete volume to pile length was very consistent, averaging about 1.38 (theoretical value being 1.31), this indicating a very slight "overbreak" of the concrete into the surrounding soil.

After completion of drilling of each pile, verticality checks were performed, with 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 1) were consistent with those obtained during the mix acceptance tests.

Table 2. Plastic Concrete UCS in MPa (from site laboratory tests in cylindrical samples)

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

4.5 In situ tests of the completed C/W

A significant number of in situ tests was conducted in the completed C/W structure. These included check holes with full core recovery, water permeability, SPT and CPT tests [2].

A total of 18 check holes were drilled in the C/W, 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. The most effective coring method proved to be a single tube core barrel, 113mm o.d., with a thick crown and no water flush. The runs were 1.50m long and the core diameter was 100mm. The holes were also subjected to Maag tests, and some of them were left open to allow the permeability to be calculated by measuring recharge characteristics following adjacent dewatering tests. 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 $1.95 KN/m^3$
- 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 (it may 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.
- Pocket penetrometer testing gave UCS values of the order of 0.30-0.50 MPa. The reason for these low results is again attributed to the disturbance of the samples due to the coring process.
- 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.

SPT tests were carried out in 12 of the check holes. The following observations were made:

- There was no evidence of soft/unhardened material in any pile, with the exception of the upper 3m (embedded however in the core material) in one pile.

- With the same exception, the SPT N values obtained were relatively uniform, being generally in excess of 30, and were indicative of very stiff/hard conditions, consistent with a UCS in excess of 400KPa.

- No systematic improvement of the backfill with age was recorded in a case where the test was repeated with a 30cm offset after an interval of over 26 months. This however may be due to the disturbance caused by the first phase of testing.

CPT testing was carried out in five holes, after the SPT tests. They also indicated that in general the plastic concrete could be classified as a very stiff sandy silty clay, with Cu $\geq$ 600MPa, and UCS $\geq$ 1.20MPa. CPT values were lower in the uppermost one meter of the piles, apparently disturbed by the previous block sampling process.

As with the SPT testing, CPT testing is best taken as an indication of general trends and confirmation that no significant anomalies (e.g., soft, unhardened zones) were encountered.

4.6 Pumping tests

The efficiency of the C/W was checked by a series of tests after completion of the structure, pumping both from upstream and downstream and recording discharge and water levels in piezometers (vibrating wire and standpipes). Evaluation of the data collected proved the satisfactory performance of the C/W [2], [4], allowing finally the closure of the diversion channel.

5 CONCLUSIONS

Two years after Papadia reservoir first filling, measured total leakages are less than 320 l/min. This value is considered low, as it corresponds to 1% of the average annual river discharge. No indications of erosion or wash-out of fines from the foundation is observed.

Measurements 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 level under the dam shell, indicating the effective operation of both the C/W, as well as of the relief wells and the drainage blanket [3],[5].

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