POST TENSIONING ANCHOR CABLES TO ENSURE FOUNDATION STABILITY OF AN ARCH-GRAVITY DAM

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ABSTRACT

Arch-gravity dams transfer load into the underlying foundations through both arching and cantilever action, requiring adequate foundation strength to withstand these forces. However, a foundation may have faults, joints or zones of inferior rock that could develop unstable rock masses under loading. The foundation rockmass at Changuinola 1 Dam comprises a series of volcanic rocks, which generally implies non-continuous joints and abrupt frequent variation in rock strength and deformation moduli across the foundation. On exposing the foundation, a relatively weak area with unfavourably orientated joints was identified at the dam toe on the right abutment. The joints in this area were found to be orientated in a similar direction to the main thrust forces of the dam, and a means to increase the sliding stability was required. This was accomplished by increasing the frictional resistance on potential failure planes by introducing post-tensioned anchor cables. This paper presents the analysis, design and detailing of anchor cables on the abutment to ensure foundation stability of the arch-gravity dam.

1. INTRODUCTION

Changuinola 1 Dam located on the Changuinola river, Bocas del Torro Province, Panama was constructed as an arch-gravity dam. The high-grade Roller Compacted Concrete (RCC) dam has a maximum height of 105m above the lowest foundation level.

Figure 1: Changuinola 1 Dam, Panama

General requirements for the design of rock foundations for concrete dams are stability against overturning and sliding, acceptable levels of differential settlement and control of seepage. The stability of an arch-gravity dam is controlled by the foundation geometry, rockmass attributes and the strength and deformation modulus of the rockmass. Geological features such as joints, faults and zones of inferior rock could develop unstable rock masses under loading.

The foundation rockmass at Changuinola 1 Dam comprises a series of volcanic rocks, and quite different strength and deformation parameters are apparent in the different formations and rock types. It was thus understood in the dam design that a significant variation in rockmass stiffness and areas of relatively low stiffness could be expected. The foundation design consequently included a continuous collection of data and ongoing foundation stability evaluation from the time of initial foundation depth determination to the completion of the dam.

2. FOUNDATION GEOMETRY AND ROCKMASS CONDITION

On completion of the evaluation and subsequent improvement of the dam foundation in the form of consolidation grouting, areas of low deformation modulus and unfavourable joint orientation still remained within the dam foundation. The varying deformation modulus values across the foundation are indicated in Figure 2.

While it has been proven via structural analysis that the dam will successfully bridge certain isolated areas with low deformation moduli, specific mitigatory measures were required on the lower right flank, in foundation blocks 21 and 22.

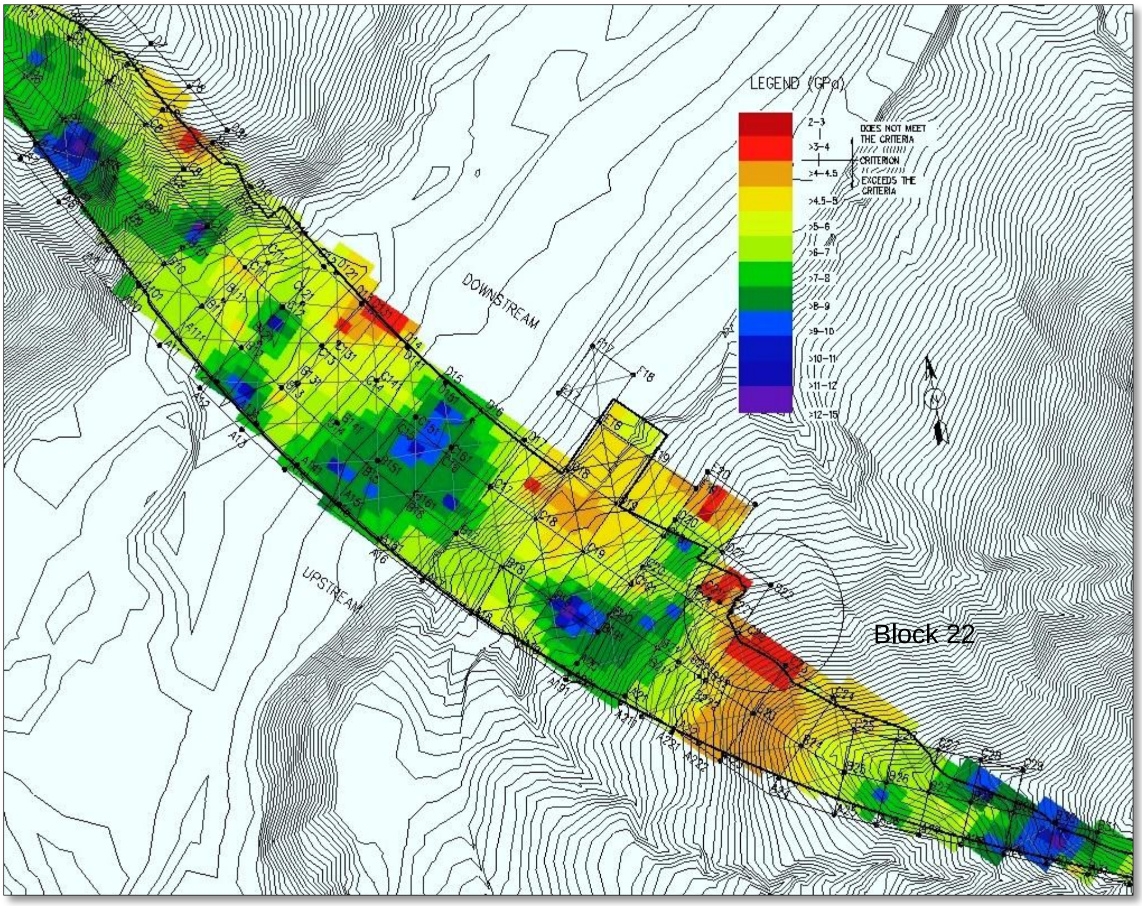


Figure 2: Deformation Moduli across dam foundation

2.1 Block 22 surface geology description

Various vertical rockmass discontinuities are encountered perpendicular to the dam axis forming vertical faces. Block 22 is characterized by a particularly high vertical face, approx. 20m high as presented in Figure 3.

The rockmass at the upstream half of the foundation comprise “relative” high strength Andesite, while the downstream portion consists mainly of coarse, medium-strong Agglomerate presenting particularly very weak foundation material (deformation modulus lower than 2GPa.) The intact rockmass strength and stability is further compromised with intrusions of weak Tuff material.

The Agglomerate matrix can be described as fine grained, dark coloured and moderately weathered. When exposed to the atmosphere, the rock surface weathers rapidly. A greenish coloured Agglomerate-Tuff matrix located more to the downstream is indicative of a high Chlorite content. The rockmass is highly weathered with calcite veining in these areas.

2.2 Block 22 surface jointing description

Generally all the major identified joints are continuous undulating fractures, filled with either highly weathered Agglomerate or Clay, infill thickness varies from 10 to 700mm. The identified joints are grouped into three main directions i.e.: steep joints which are either dipping to the west or north and the sub-horizontal joints which are dipping towards the south.

Two unfavourably orientated continuous joints were identified at the downstream end of Block 22. Figure 3 presents the location of the identified joints. The joints form potential failure planes which are parallel to the direction of the major arch thrust force. The joints are described as follows:

- Joint 27 - Dip 30°, dip direction 200°
  - 10 – 30mm wide filled with soft brown Tuffic Clay
- Joint 31 - Dip 20°, dip direction 260°
  - 150 - 400mm wide filled with highly weathered Agglomerate.

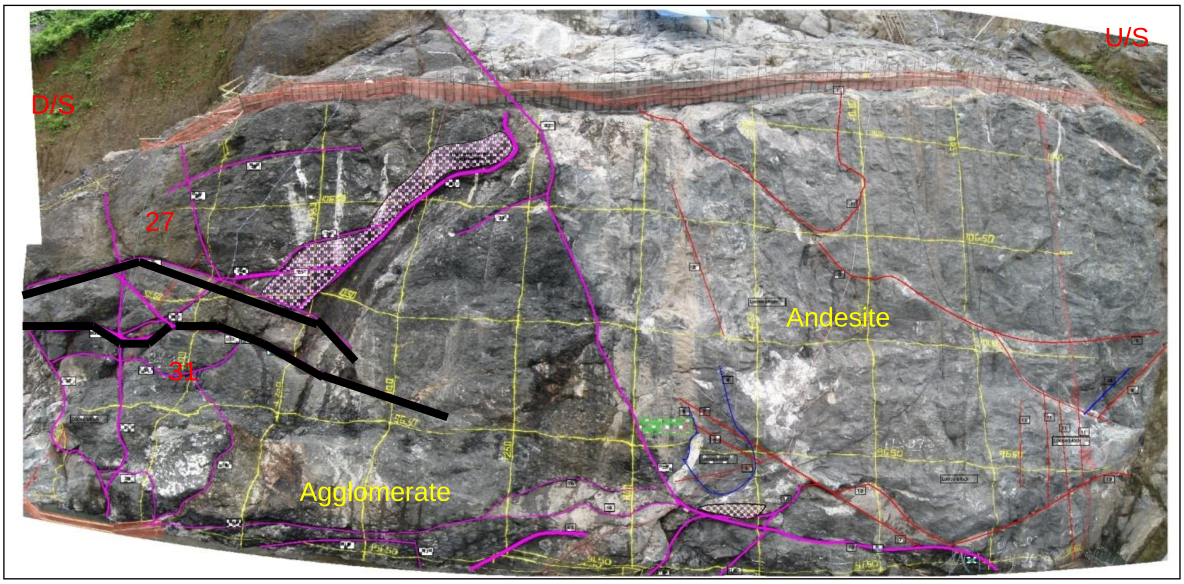


Figure 3: Elevation on Block 22

3. FOUNDATION STABILITY ANALYSIS

3.1 Material properties used in analysis

When the weak area at the downstream toe of Block 22 was identified, specific attention was given to field mapping of the identified weak area. The rockmass attributes (GSI, rock strength, RQD, joint spacing and joint condition) as reported by field geologist, were evaluated and analyzed. Subsequently the program Roclab was used to determine the material properties presented in Table 1. (Changuinola Civil works. 2011. Foundation completion report)

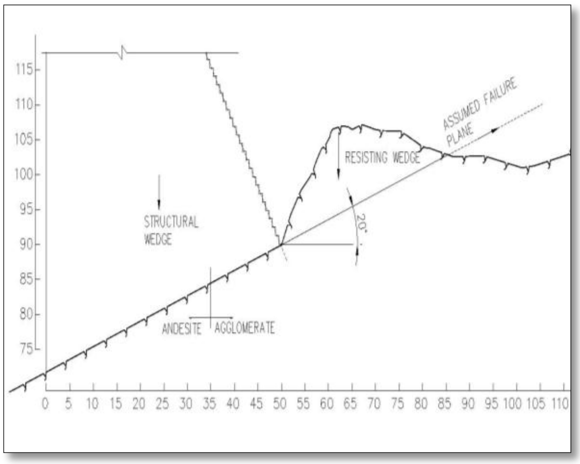
Table 1: Material properties at downstream end

Joint material description	Cohesion (c) (kPa)	Internal friction (φ) (°)
Parent material (Weak Agglomerate)	480	40
Joint material (Crushed rock/clay infilling)	200	30

3.2 Analysis on a specific plane

Arching action implies that significant thrust forces are directed into the foundation on the lower flank, and specifically on the downstream portions of these areas. For the purposes of evaluating the stability of the foundation, forces imposed on the foundation were derived from finite element analysis of the dam. Whilst stability across the whole foundation is paramount, the large arch thrust forces coincided with the location of the toe in the weak Agglomerate. Accordingly the stability analysis concentrated on the impact at the downstream toe area and the two critical joint planes were analyzed in detail.

Although the identified joint sets represent a relatively localized phenomenon, they could conceivable represent a continuous plane beneath the dam. The resultant between the force in the direction of the plane and perpendicular to the plane was determined and the sliding stability analysis performed in the direction of the resultant.



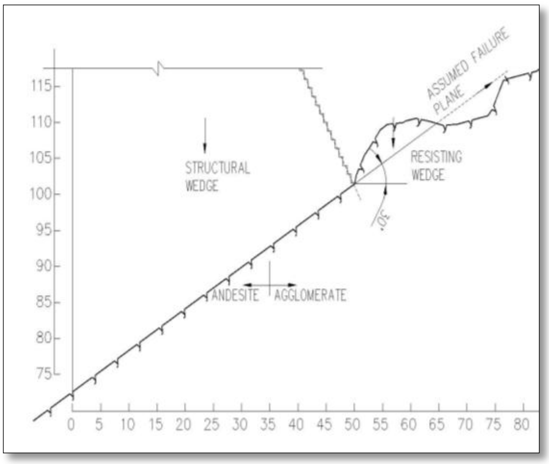


Figure 4: Critical sliding plane on joint 31

Figure 5: Critical sliding plane on joint 27

A multiple wedge analysis was used to determine the sliding factor of safety along the critical sliding planes. Typical sections along the identified planes in the direction of the resultant as indicated in Figure 4 and Figure 5 were used to investigate the extent of resisting wedges.

3.3 Analysis results

Taking into account the conservative 2D analysis on the sliding planes, it was accepted that a safety factor of 1.5 under normal load conditions could be adequate for the stability of the dam foundation. However the factor of safety achieved on the sliding planes formed by joint 27 and joint 31 was not considered adequate. Analyses results indicated a safety factor as low as 0.92 on the sliding plane formed by joint 27.

Even though consolidation grouting was specified to generally improve the highly weathered Agglomerate rockmass properties, this was considered inadequate in the light of the potential for fractures in the joint planes identified. Various scenarios of improvement, including the provision of stability masses, and further excavation to remove material above the potential sliding planes were evaluated. Due to programming and resource constraints, the only feasible solution proved to be to increase the frictional resistance on the potential sliding plane. This could be accomplished by either increasing the frictional resistance on the planes, or by directly opposing the destabilizing forces. The latter proved geometrically impossible and it was considered that the most effective means to achieve this was to introduce the necessary resistance forces into the foundation by using post-tensioned cable anchors.

4. DESIGN AND ORIENTATION OF CABLES

The proposed cable anchors are primarily orientated in a direction to increase the normal force and subsequently the friction on the identified critical planes. This necessitated a concrete anchor block at ground level from where the anchors could be stressed, accepting an anchorage at a safe distance below the potential sliding plane. The resultant force on each sliding plane is the critical force, thus the foundation must be stable on the subsequent resultant plane of the two identified joint planes. Accordingly the anchors were designed to increase the normal force on the resultant planes. Analysis indicated that the resultant force on joint plane 31 was greater than the resultant force on joint plane 27. Subsequently the anchor cables were designed normal to joint plane 31 and the projected normal force was used in analysis at joint plane 27 as indicated in Figure 6.

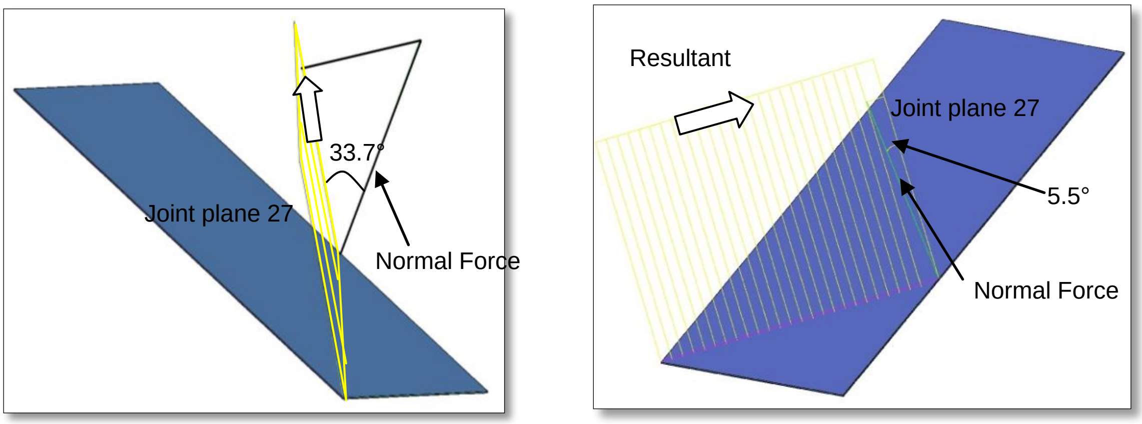


Figure 6: Projected normal force on joint plane 27 in a) upstream-downstream, b) diagonal direction

The cable anchors were installed at 18.7° from the vertical and 107° from north, normal to the resultant joint plane of joint 31.

5. DESIGN OF FOUNDATION STABILITY STRUCTURE

5.1 General

Initially 23 anchors with 20 strands/anchor were specified. However, logistics proved to be a determining factor. The EPC contractor would have had to import a drilling rig that could drill the required anchor hole diameter which proved too costly. It was also his preference that the complete anchor be manufactured off-site and shipped to site in containers. Accordingly the maximum number of strands which could be coiled and shipped in a container was dictated by the supplier to be 12 strands/anchor. Consequently 43 anchors with 12 strands/anchor were included in the foundation stability structure.

5.2 Reinforced concrete anchor block

Taking into account that the foundation at the toe of Block 22 was highly weathered it was necessary to cover the whole affected area with concrete to prevent further weathering. Accordingly a reinforced concrete anchor block was designed through which the anchors could be installed. This block resembles the step configuration of an RCC dam to effectively blend the anchor block alignment into the RCC dam as presented in Figure 7 and Figure 8.

After studying and evaluating the geological data of the affected area, it was quite apparent that the reinforced concrete anchor block would be founded on a “relatively” elastic foundation. However, with consolidation grouting specified, isolated areas with either a high or low bearing capacity could be expected and this was included in the analysis and design of the post tensioning system.

The anchor cable stressing sequence was specified to commence from the middle of the anchor block, outwards towards the sides to prevent excessive bending stresses at any particular area. Considering the enormous forces applied to the anchor block and the “relatively” elastic foundation, the anchor block was heavily reinforced to safely distribute the loads into the concrete and subsequently into the foundation.

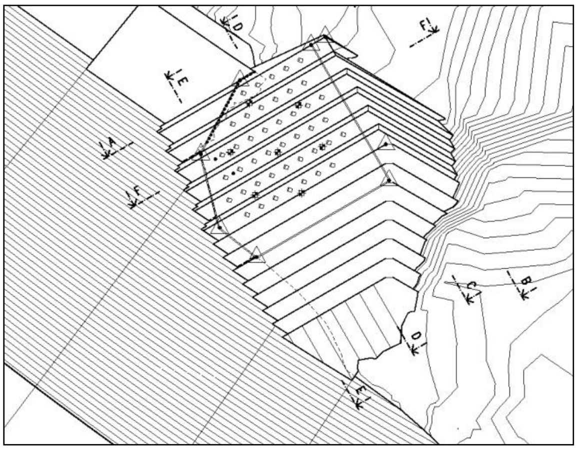


Figure 7: Plan layout of anchor block

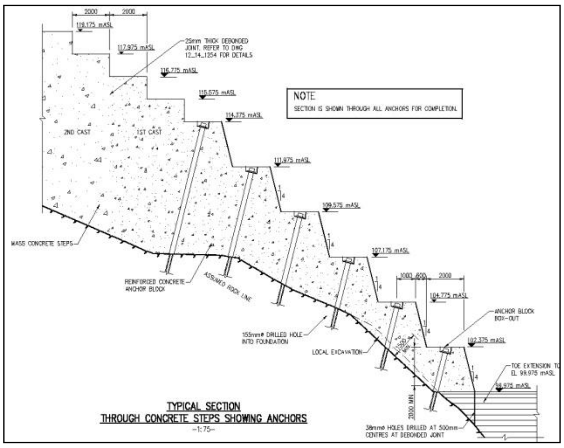


Figure 8: Section through anchors

6. DETAILS OF ANCHOR CABLES

6.1 Anchor length

To ensure adequate force transfer onto the sliding plane, the unbounded length (anchor length) of an anchor is determined by the depth and orientation of the failure plane and/or the location of competent rock capable of resisting the anchor force. Therefore, after evaluation of various core holes drilled in the affected area and identification of the failure planes, the anchor cables lengths required varied from 33m to 37m.

6.2 Anchor details

The cable anchors comprise post–tensioned tendons which are installed in drilled holes and the entire bond length (8m) is located in competent rock below the failure planes. The anchor force is transferred to the rock by the bond between the grout and the rock.

The following anchor specifications were used:

- Strands to be 15.24 mm diameter, low relaxation steel, elastic modulus = 198 KN
- Characteristic breaking load per strand/tendon = 260.7 KN

The cable anchors remained unbonded in the free stressing length allowing the anchor to be checked and re-tensioned at any time. To prevent corrosion, protection in the form of HDPE sheaths around the tendons was provided in the free length of each strand. An outer HDPE sheathing was furthermore included in the system to prevent any possible corrosion attack. In the anchorage zone this was configured to allow bond forces to be transferred to the surrounding rock.

Tendon spacers were used inside the HDPE sheathing to separate the individual strands and ensured adequate grout cover to all the strands.

As elastic deformation of the foundation was anticipated during and after loading of the anchors, a re-stressable anchor head was specified.

Typical cross sections and a section through the anchor centerline are presented in Figure 11.

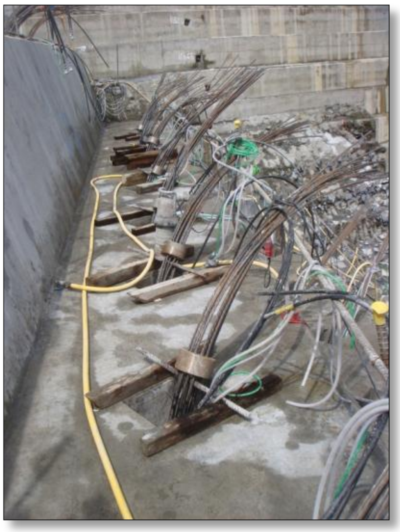


Figure 9: Anchor cables installed

Figure 10: Installing anchor cables

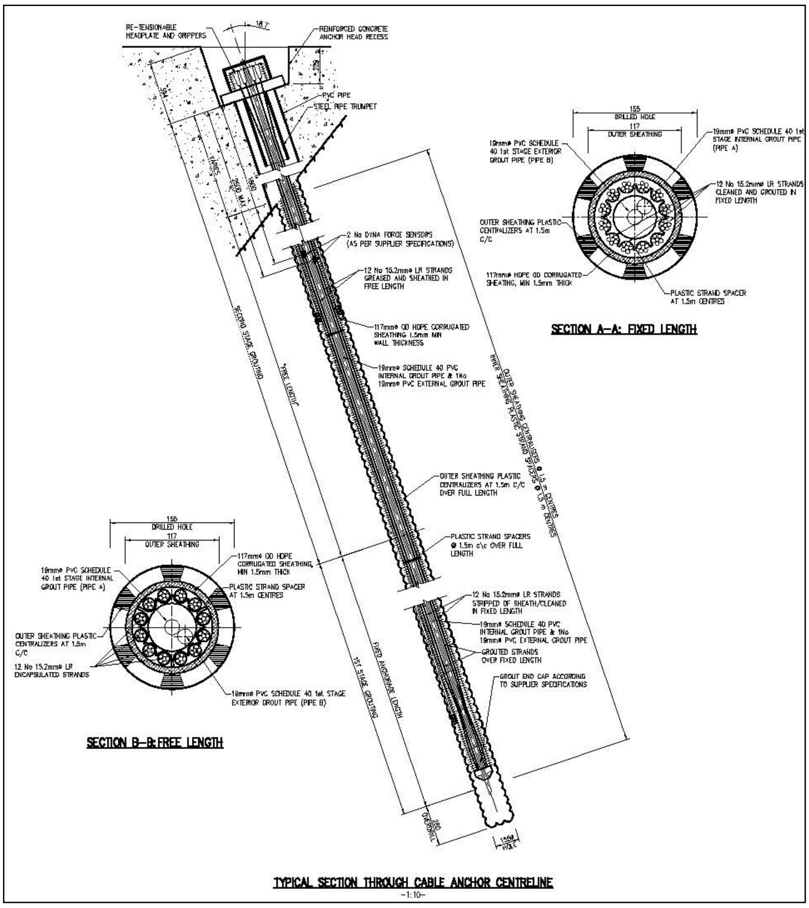


Figure 11: Cable anchor details

7. DRILLING AND GROUTING

7.1 Drilling procedure

To preclude cutting of reinforcement during the drilling procedure, PVC pipes were cast into the concrete anchor block. Initially a 95mm diameter pilot bit was used to drill through the PVC pipe over the entire length of the anchor hole. Expecting fractured and weathered rock, Lugeon tests were specified after drilling of the anchor holes to confirm the integrity of the rock. A comprehensive water test was executed on completion of the drilling. When Lugeon values above 1 were encountered the hole was grouted to ensure water tightness in the anchor hole and improve the rockmass surrounding it.

The holes were grouted under a pressure of 250kPa with the packer placed at the top of the hole. Initially grouting was done with 1:1 water: cement mix by mass. Grout consistency was thickened to 0.6:1 when the grout take exceeded 100 liters/m.

After the grouting, the pilot bit was used to re-drill the hole and another comprehensive water test was performed until acceptable Lugeon results were obtained. The anchor hole was reamed to 155mm diameter with a percussion drill after a successful water test.

7.2 Grouting of the anchors

In accordance with recommendations by the anchor supplier, the whole length (except space in and immediately below the anchor block) was grouted before tensioning commenced.

Grouting was performed through 20mm diameter PVC pipes both inside and outside of the HDPE sheathing to the top of the free length. Taking into account that the tendons in the free length are sheathed, re-tensioning will still be possible at a future date although some deformation of the installed grout outside the sheath will occur.

8. STRESSING OF ANCHORS

The stressing of the tendons commenced only after the concrete anchor block and the grout attained the design compressive strength and prior to filling of the dam. The stressing sequence is presented in Figure 12.

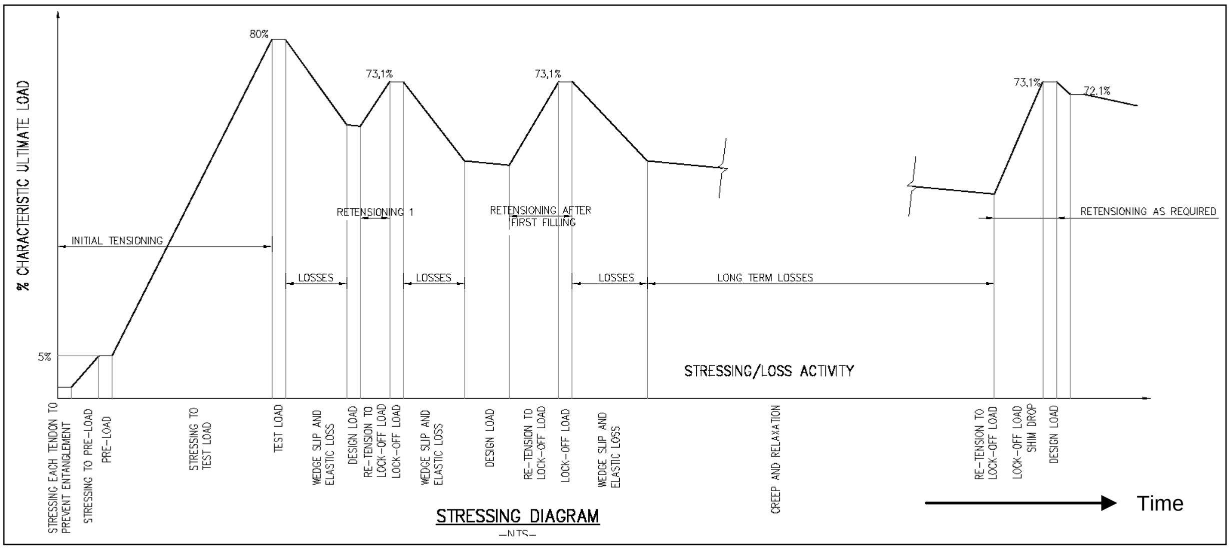


Figure 12: Stressing sequence

Figure 13: Stressing of cable anchors

Expecting compression of the foundation material with consequent settlement of the anchor block the anchor heads were surveyed prior to and immediately after stressing of the anchors to establish the elastic shortening of the compressed rockmass.

Initially a small load was applied to each tendon to prevent entanglement of the tendons, after which the load was increased to a preload of 5% of the ultimate load. The load was then increased gradually to the specified testing load (80% of ultimate). All tendons were checked for excessive sequential losses in the same order as initially stressed after lock-off. When the lift-off load was found to be less than 68% of the ultimate load, the tendons were re-stressed to 73.1% of the ultimate load.

Re-stressing is specified after first filling of the dam, where after Dynaforce Elasto-Magnetic (Dywidag-Systems International) sensors installed on certain cables will be monitored on an ongoing basis. Should the tendon force fall below 62% of the ultimate, the anchors will be re-stressed. A ten year regime of restressing each anchor was specified.

9. MONITORING

Other than the Dynaforce Elasto-Magnetic sensors installed on the cables, two Inclinometers were installed in the anchor block to monitor any movement of the critical failure planes.

The Inclinometers are orientated so as to perpendicularly intersect the critical resultant plane. Tilt sensors have further been installed at the elevation of the expected failure planes to monitor any isolated movement on these planes.

10. CONCLUSION

The bearing area and shape of the foundation for arch-gravity dams are configured to ensure that the underlying rockmasses have adequate strength, bearing capacity and sliding resistance to withstand the arching actions induced in them. In the case of the Changuinola 1 Dam the adequacy of the rockmass at the downstream toe at a particularly sensitive area was found not to be sufficient to withstand the loads induced. The potential for transmission of the loads to the abutments and complete arch action was consequently compromised. The provision of anchor cables on the downstream toe served to increase the load bearing capacity by preventing wedge failure at various potential failure planes.

Anchor cable systems are used regularly in combination with arch dams when doubt exists as to foundation integrity. At Changuinola 1 Dam, the incorporation of anchor cables on the right abutment proved to be a cost-effective solution to ensure foundation stability.

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