SAPROLITE FOUNDATION PORE PRESSURE RESPONSE AND INFLUENCE INTO DAM CONSTRUCTION STAGING

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ABSTRACT

The design of a dam is based on the specific characteristics and the climate conditions at a given site. However, the design process will continue throughout construction to assess compliance with the design intent fed by monitoring of the foundation, dam fill performance and the evaluation of good construction practices. This paper presents lessons learnt from the design and performance monitoring of a dam constructed on a thick residual saprolite profile. The application of an observational approach for the dam construction staging and hold points for stability at each dam construction stage are discussed based on pore pressure measurements and slope stability analyses provided a rapid rate of raise was required due to the high rainfall climatic setting.

1. INTRODUCTION

A weathered saprolite foundation consists of soil-like materials exhibiting the fabric of the parent rock, present as relic structure. Under shearing, saprolite typically responds as a fine-grained soil with undrained response under rapid shearing. Weathered soils are typical in tropical high rainfall sites.

A dam in this foundation setting will require to be designed and build in stages to promote strength-gain due to consolidation. In contrast, the dam will be builtrelatively fastto compete with the fast rate of rise of the impoundment, if water is allowed to pond during construction,to meet freeboard. Furthermore, the stability of a fast raising dam should comply with the stability criteria established based on the site-specific foundation conditions and dam fill materials. Mitigation measures should be in place provided something unplanned occurs under such conditions.

This paper summarizes the design approach and field observations from one dam constructed at a high rainfall sites in South America and summarizes lessons learnt and challenges encountered.

2. SITE CONDITIONS

This sitehas an average annual precipitation over 3000 mm, raining most of the days. The annual evaporation is about 1000 mm, which results in a positive net surplus water setting.

The site has a unique steep mountainous terrain with steep and sharp ridges. Weathering of the granodiorite and monzonite bedrock saprolite is thicker on the ridges up to 45 m depth and thinner at the valleys ranging from 10 m to 20 m thick. Saprolite overlies a transition zone to fresh bedrock.

The construction materials or methodologies used for this dam are not discussed in detail in this paper; however, it is noted that the challenging of getting proper granular materials for filter and for a strong well-draining shell to support the fine-grained seepage barrier are crucial for the success of any project under such extreme climate conditions. Planning for construction equipment and material sources ahead of time are key to avoid delays during construction.

3. ANALYSIS FRAMEWORK

Table 1 shows our assessment and statement of the problem and potential key issues. This brainstorming is typically conducted prior to the design process. Table 2 shows a summary of the known and unknowns for this project, which assisted to setting key questions prior to the analyses.

Table 1 : Summary of the design approach.

Item	Step	Description
1	Establish the problem,objective and limitations	• Thick saprolite foundation with potentially weak undrained shear strength and slow dissipation of excess pore pressures. • High rainfall inflow, resulting in a fast rate of riserequiring a fast rate of raise for the dam. • Limitations may include procurement process and access to the areas for site investigations.
2	Collect the informationand formulate the concept.	• Site investigations including mapping, 60 test pitting, 25 drill holes with core logging, 30 infiltration tests, 25 Standard Penetration Testing (SPT), 25 Vane Shear Testing (VST) in drill holes and 55 hand-held VST and 40 falling-head tests. • Sampling of disturbed and undisturbed samples (thin-walled tube and block samples) for index testing (i.e., particle size distribution, Atterberg limits, specific gravity, water content, etc.), 18 Triaxial Consolidated Undrained (TXCU), 3 Direct Simple Shear (DSS), and 6 one-dimensional consolidation tests.
3	Conduct analysis andestablish main controlling factors and provide design alternatives	• Produce cross-sections and a dam profile with information from Step 2, including simplified drill hole logs, SPT profile, water content profiles, and key information from site observations. • Establish most-likely and worst-case scenarios based on a summary of the laboratory and in-situ data in lieu with the geotechnical sections and case histories. • Establish minimum undrained shear strength of the foundation and response to shearing. • Establish the range of consolidation coefficients to estimate dissipation rates of excess pore pressures for stability analyses. • Establish potential excess pore pressure coefficients (B-bar) for a proposed construction staging. • Estimate Factor of Safety for various scenarios.
4	Optimize the design with observations during construction	• Identify Key Performance Indicators (KPI) for monitoring during construction and after construction into operations. • What parameters do we need to monitor? and what actions are required if observations deviate from the design assumptions? for example: pore pressure and fill rate of raise.

Table 2 : Known/unknown evaluation prior to analyses

Item	Known	Unknown
Known	**Facts** 1. Saprolite thickness and extent 2. Saproliteundrainedshearstrength,moisture profile, coefficient of consolidation and hydraulic conductivity. Fast dam rate of raise.	**Questions** 1. How fast the dam will be raised? 2. How fast the foundation saprolite will dissipate the excess pore pressures? 3. How much strength will be gained with progressive consolidation?
Unknown	**Intuition** 1. Excess pore water pressure in the foundation may dissipate and foundation will gain strength with time. 2. If no dissipation is observed, construction should be halted and/or either a flatter slope may be used, or an additional buttress berm will be required to achieve the design FOS.	**Risks** 1. No pore pressure dissipation, thus the stability will be controlled by the minimum undrained shear strength of the foundation. 2. No time to build a buttress berm. 3. No pumping capacity to slow impoundment rate of rise.

4. DAM DESIGN

4.1 General

The dam is classified as an Extreme consequence to failure based on the Canadian Dam Association guidelines for mine waste deposits (CDA, 2007,2014).

Relevant to the objective of this paper, the design basis for this project included a minimum storage requirement for the installation of a reclaim barge and a tight deadline to achieve the dam elevation for early commissioning. The design criteria included storing the Inflow Design Flood (IDF) event without release to the environment, a minimum freeboard of 1 m (vertical distance between the IDF level and the minimum dam crest elevation), and a minimum factor of safety of 1.5 during construction. A factor of safety higher than 1.3 (CDA, 2007,2014) was established due to the uncertainties of the foundation conditions, construction methodologies and final conditions of the dam fill.

The minimum dam crest elevation prior to operation was established for a30 m height dam to provide storage for the first year of operation and to safely store the IDF event while maintaining the minimum freeboard allowance.

The dam does not have an emergency spillway; therefore, the dam rate of raise is more sensitive to the flood storage requirement rather than the assumptions made for the tailings deposited density. The IDF was set as a larger volume than the required for the Extreme classification of the Probable Maximum Flood (PMF).A contingency margin was agreed to provide safe storage and to reduce risk of overtopping during construction.

4.2 Foundation characterization

Table 3 shows the adopted material properties for the saprolite foundation after the foundation investigations. Figure 1 shows the summary plots of index properties with depth and consolidation and shear strength test results.

Saprolite is a low plastic sandy silt that becomes coarser (higher sand-sized particle content) below 12 m depth to a silty sand. The saprolite average specific gravity is 2.6 with Kaolinite as the predominant clay mineral with no presence of swelling minerals. Moisture content decreases with depth from about 60% to an average of 20%. A difference of less than 4% between both theoven-dry 50°C (Blight, 1997) and air-dry compared to oven-dry to 110°C (ASTM D2216), indicated low influence of structural water in material properties. The average liquidity index ranges from 2 to 0.5 with depth.

Saprolite has low to medium hydraulic conductivity as seen ininfiltration tests and falling-head tests at the installed Casagrande piezometers. The compressibility of the saprolite is low to medium and shows strain hardening with cumulative vertical strains ranging from 12% to 22%. The yield stress, estimated from 100 kPa to 400 kPa, is defined as the stress to which the chemical bond or relic structure created or left in place during weathering is broken. Its relationship to the engineering behaviour its comparable to a pre-consolidation stress.

Saprolite strength increases with depth as observed in SPT and VST testing conducted at 1.5 m intervals up to 20 to 52 m depth (see Figure 1). Saprolite contracted under rapid shearing in the triaxial consolidated undrained (TXCU) test for stresses up to 1000 kPa. TXCU testing showed that the peak undrained shear strength ratio decreased with confining stress from 0.70 (stresses less than 200 kPa) to around 0.3 (stressesgreater than 800 kPa). The principal mode of deformation for this case is through the upper, weaker, saprolite in a horizontal direction, which is better defined by a direct simple shear (DSS) test. The DSS on trimmed saprolite from block samples showed that after yielding, the peak undrained shear strength ratio is 0.28 for the range of stresses tested. See Figure 1.

The effective strength of the saprolite can be modelled by an effective friction angle of 30 degrees and a cohesion intercept of 10 kPa. However, the dominant shear mode will be undrained.

Table 3 : Typical Material Properties of the saprolite foundation

Item	Unit	Saprolite Foundation	
		0 to 12 m (Upper)	12 to 20 m (Lower)
Description		Sandy silt or silt (ML); low plasticity; varies from soft to very stiff with depth; colour varies from reddish brown, pink, and white to yellowish brown or light brown, with depth. Parent rock structure is evident. Below12 m saprolite is generally described as silty sand (SM)	
Percentage of fines	%	Up to 75	Up to 40
Moisture content	%	20 to 60	20 to 30
Unit weight	kN/m^3	17.8	18.4
Coefficient of compressibility (m_v)	kPa^{-1}	1x10^{-4} to 4x10^{-3}	
Coefficient of consolidation (C_v)	cm^2/s	6x10^{-3} to 5x10^{-2}	
Hydraulic conductivity	m/s	1x10^{-8} to 1x10^{-7}	
Minimum undrained shear strength	kPa	30 to 160	>80
Undrained shear strength ratio	-	0.28 to 0.85	
Range of N-value from SPT	blow count	4 to 15	>15
Liquidity index	-	1	< 1

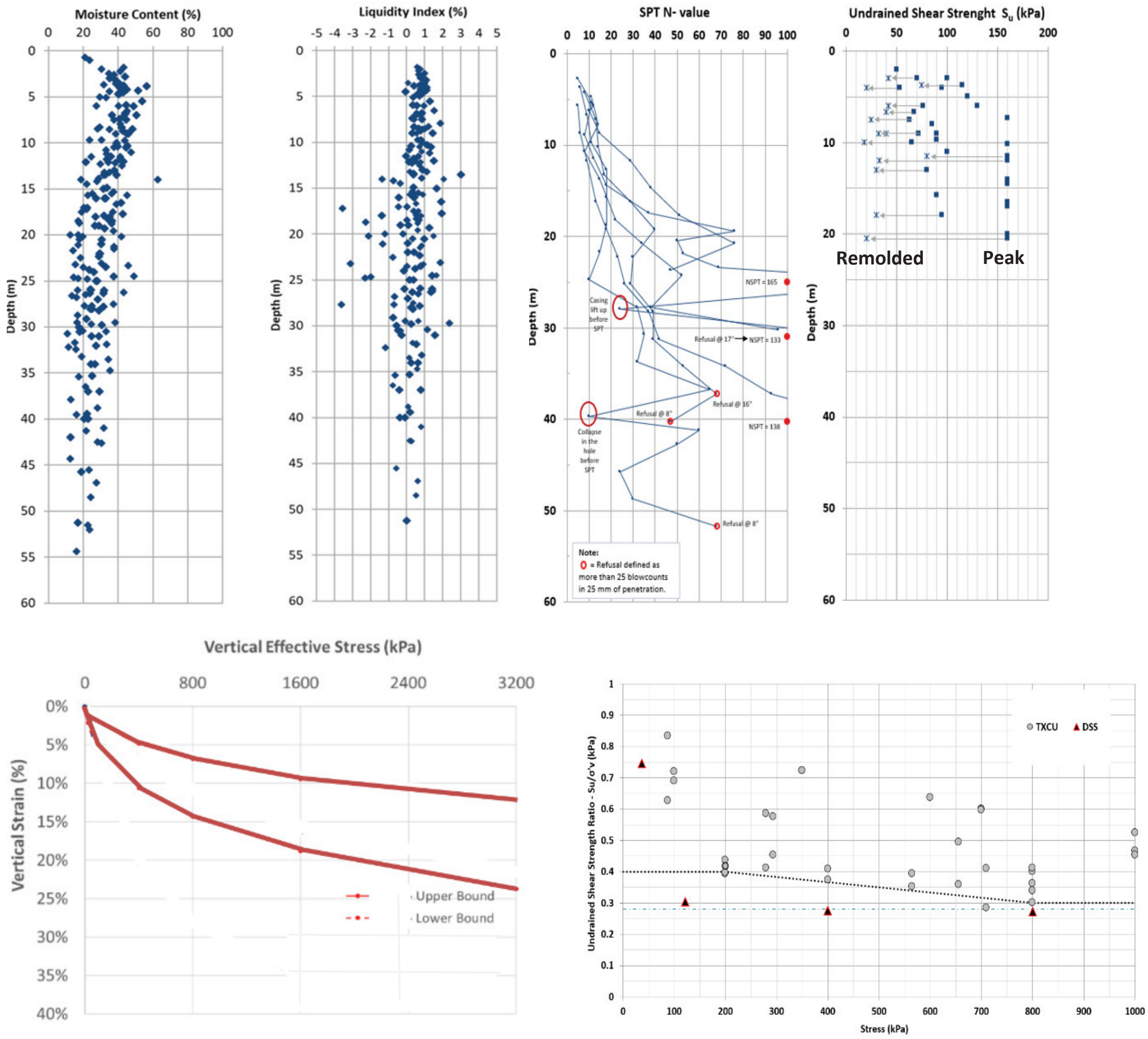

Figure 1 : Summary of saprolite properties. Top: Moisture profile, liquidity index, SPT and VST test results with depth; Bottom left, range of vertical strain from oedometer test results, right, summary of shear strength

4.3 Dam design section and staging

The dam cross-section along the main valley used for the analysisis shown in Figure 2 with a simplified foundation model including the upper and lower saprolite, the transition zone and bedrock. The dam is 30 m high (H) and sits on a v-shaped valley with an aspect ration (length: height) of 4.

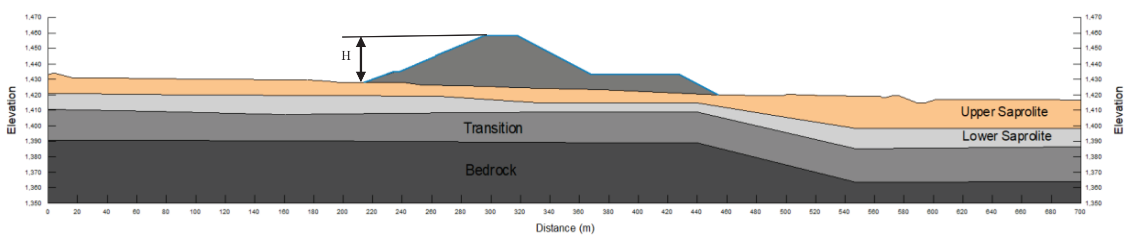

Figure 2 : Scheme of the dam configuration.

The design approach for the upstream slope, were the finer-grained materials are needed for seepage barrier, was:

1. Hold Point No 1: Build the dam with a 3H:1V slope for the first 6 m (0.2H) to allow the contractor to gain knowledge on construction materials and monitor instrumentation.
2. Hold Point No 2: Raise the dam with a 2.5H:1V slope to about 0.4H, which is the height compliant with a FOS of 1.5 assuming no excess pore pressure dissipation and the minimum undrained shear strength after ground preparation of50 kPa for the foundation.
3. Building above Hold Point No 2 was subjected to the observations of foundation excess pore pressure dissipation from installed instrumentation. The strategy was:

(a) If excess pore pressure dissipated during the initial construction, then Hold Point No3 would be set based on the peak undrained shear strength ratio of 0.28 with a varying and the actual B-bar estimated during the construction.

(b) If no excess pore pressure was observed, then construction should either be halted to allow pore pressure dissipation,or the stability should be increased by building an upstream buttress berm towards uprising ground on the upstream side of the dam. This would allow to continue raising the dam and would provide time for pore pressure dissipation. The volume of thisupstream berm was estimated in advance and the specifications for the fill were discussed prior to the construction.

Figure 3 below shows the influence varying excess pore pressure coefficient (B-bar) to the FOS for different heights relative to the final height, H.As seen, a higher B-bar will restrict the raise of the dam to a lower elevation. As pore pressures dissipates, the dam could be built higher without reducing the target FOS. This graphwas produced using the design values and material properties prior to the construction ofthe dam. Therefore, the analyses did not include the actual material strengths achieved during construction.

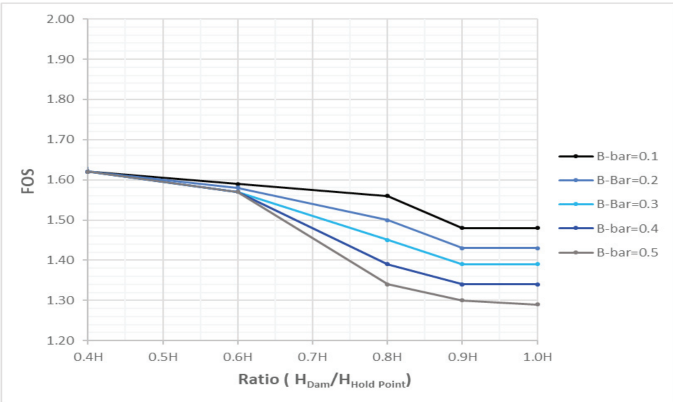

Figure 3 : Summary of FOS for different B-bars and dam heights relative to the final first raise.

For the downstream slope, the staging was less complex due to the use of granular materials for the construction of the dam shell and the presence of the underdrainage system that assisted with foundation pore pressure dissipation. The overall approach was to build the lower base of the dam towards the downstream side,so a large platform is built first to load the foundation. The downstream buttress berm was always built before raising the crest. A schematic of this staging is in Figure 4.

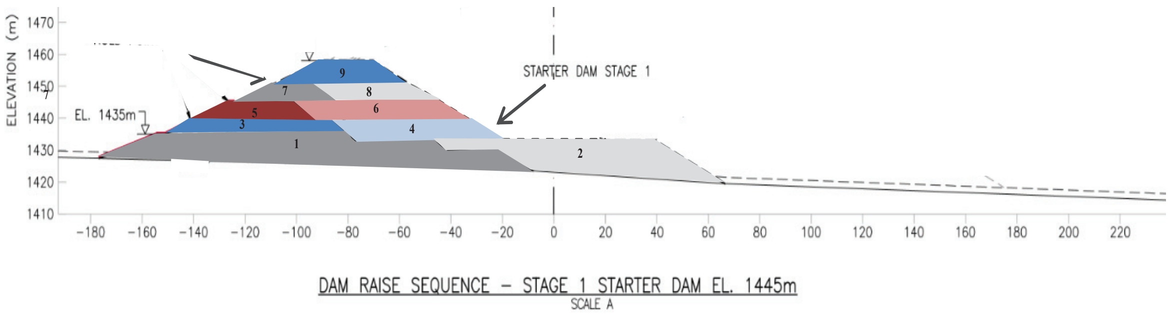

Figure 4 : Schematic of the proposed downstream raise staging.

5. DAM PERFORMANCE

5.1 General

Figure 5 shows the cycle of performance and design optimization that was used in this project. The performance of the dam was monitored via instrumentation. Additionally, the Quality Control and Assurance (QC/QA) activities allowed to record the actual material properties that confirmed or assisted to adapt the design assumptions. The data collected was reviewed against the design assumptions of most likely and worst-case scenarios following the Observational Method (Peck, 1969). The data was then used to optimize or adapt the design. In this case, the adaptation was the construction of an upstream buttress berm. Safety of the structure and personnel working on it was priority for the cycle and no decisions was taken without considering this factor.

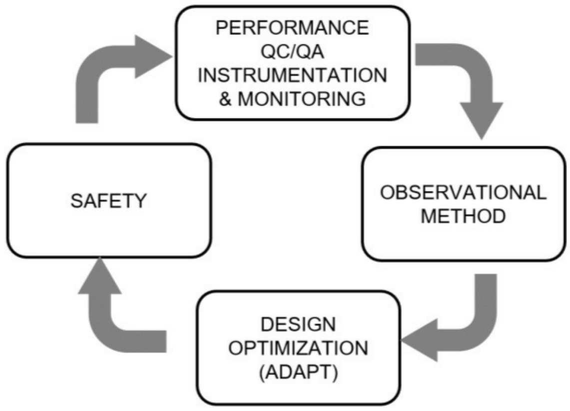

Figure 5 : Performance cycle used during the dam performance assessment.

The performance of the dam was monitored with 16 vibrating wire piezometers installed in the locations referenced in Figure6. Note that only the piezometers relevant to this paper are shown.

A total of 10 piezometers were located within the foundation to monitor the excess pore pressure during fill placement and 6 within the dam fill to evaluate pore pressures for stability analyses. Routing of the piezometer cables was challenging as the cables should not cross the seepage barrier component of the dam. Cables were extended laterally towards higher ground at common locations for data logger installation.

The ground level was surveyed at the time of installation and was recorded continuously, minimum daily during active construction, to record the change in vertical stress for a given change in pore pressure recorded from the piezometers.

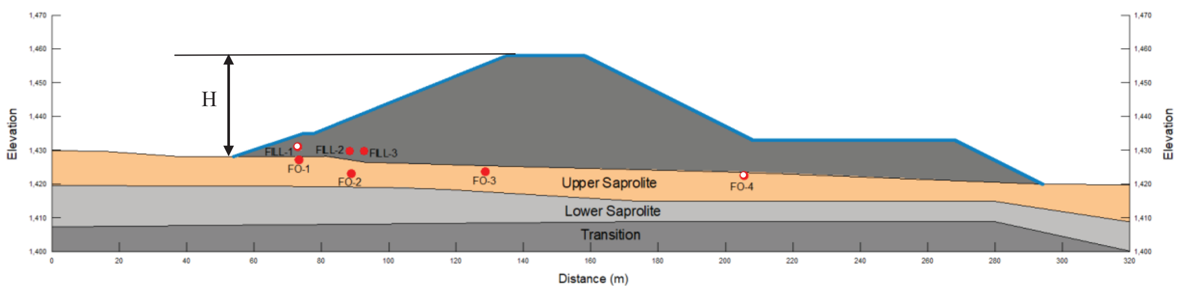

Figure 6 : Schematic of location of instrumentation for dam monitoring.

5.2 Construction observations and hold points

After clearing and grubbing, foundation was prepared to an undrained shear strength near or greater than 50 kPa. Typically, greater strength was encountered after stripping to a firm to stiff foundation.

Due to the tight schedule, climate conditions and overly wet fine-grained materials, the core of the dam achieved less strength than required for the design on the lower third of its height. This had an influence in the results shown in Figure 3 (although the failure mode is controlled by the foundation strength, the FOS is reduced when the failure mode crosses the core). Other material strength met or were close to the design assumptions and did not change the approach for the downstream slope.

Figure 7 below shows the piezometer response to fill placement and to water ponding for the construction period. As shown, the foundation piezometers responded to the fill placement and dissipated at a slow rate but with a continuous rising trend. The fill piezometers reacted faster. Piezometer Fill-3 showed a high rise in pore pressure that triggered a slow down in the construction on the upstream side of the dam at day-125. This allowed pore pressures to dissipate in the fill and steady at the foundation. This slow down also triggered the need of an upstream berm to avoid delays.

Figure 8 shows how the coefficient of pore pressure varied with time for the piezometers located beneath the upstream slope for the foundation and the fine-grained fill piezometers. The foundation piezometers dissipated from a B-bar of 0.5 to about 0.2 after 150 days. The piezometers at the fill showed higher response, but faster dissipation to B-bar from 0 to 0.15. Piezometer Fill-03 was closer to the drainage system and showed faster excess pore pressure dissipation.

Hold Points No 1 and No 2 were not modified from the original plan, but the need for an upstream support was raised following the slowing down on fill placement on the upstream side of the dam (Hold Point No3). The construction observations that triggered the need for an upstream berm were:

1. Fine-grained materials achieved a lower strength than assumed in the design for the lower third of the dam. This fact added to the high pore pressure response triggered the need for an additional buttress supporting the upstream slope to mitigate bulging of the dam if the dam were to be built higher without an upstream support for the slope.

2. Rate of raise for the dam was from 0.02m/day to 0.8m/day, faster than assumed due to tight construction schedule and deadlines. Due to climate conditions, the dam had to be raised faster during periods of "low rainfall", which required modifications to construction methodologies outside of the subject of this paper.
3. Pore pressure dissipation was observed from foundation piezometers and was within the design assumptions of coefficient of consolidation when back-analyses were performed; however, dissipation was closer to the worst-case scenario rather than the most likely, which required a slower rate of raise. B-bar ranged from 0.2 to 0.5 at the foundation. Excess pore pressure coefficients were higher within the fine-grained dam fill but dissipated faster when closer to the drainage system.Refer to Figures 7 and 8.
4. Water started to pond on the upstream side of the dam for construction and water management reasons. The rate of rise was between 0.1 m/day and 0.85 m/day, which was almost equal to the rate of raise. This also accelerate the need to raise the dam faster. Enough pumping capacity was allocated eventually to maintain a steady water level about 0.50H, which also assisted with stability.

The upstream berm was constructed to about 0.10H and about 50 m wide and provided enough buttressing to raise the dam to 0.8H provided B-bar was below 0.3 (Hold Point No4). The additional water support allowed the dam to be completed to the required target crest elevation. All the above information was used to review the dam sequencing following the steps set during the design phase.

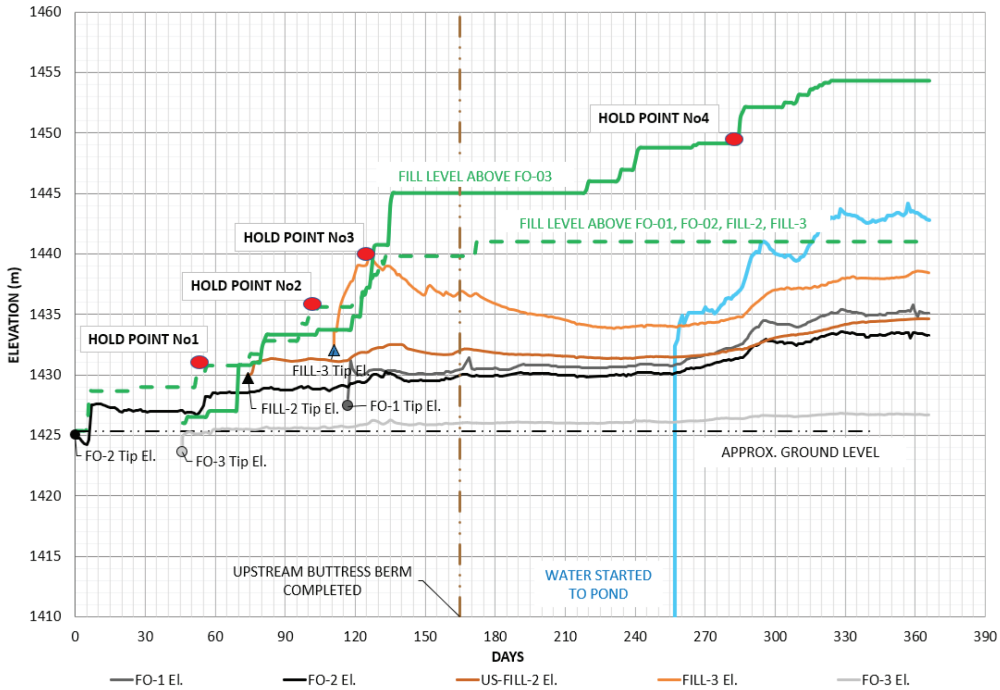

Figure 7 : Pore pressure response compared to fill and impoundment rate of rise.

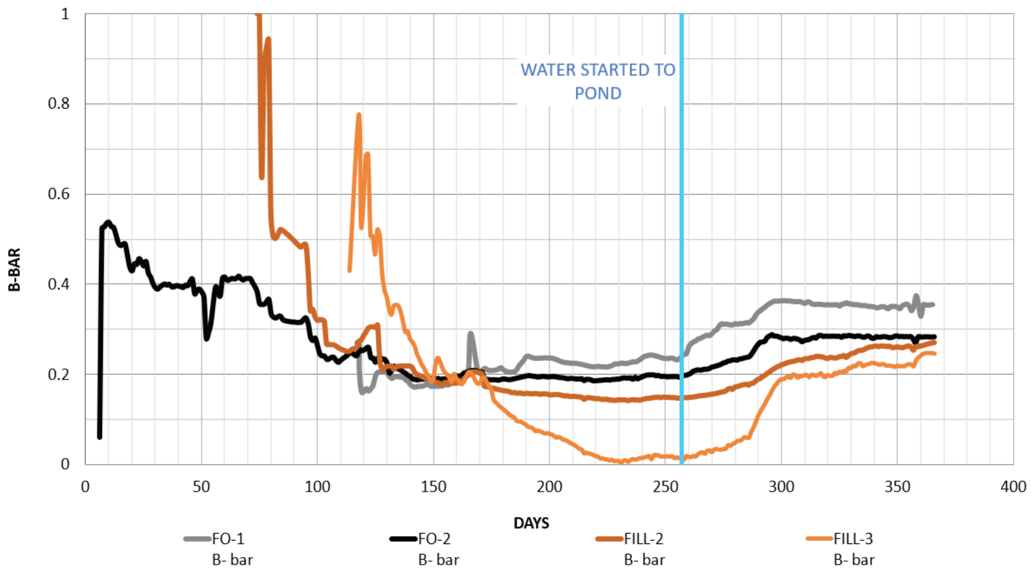

Figure 8 : Excess Pore pressure Coefficient versus time.

6. DISCUSSION AND RECOMMENDATIONS

The information in this paper showed that the design does not stop at the detailed design phase and that unknowns encountered during the design and site investigation phase should be monitored during construction and revised against design assumptions to confirm that mitigation or adaptations to actual site conditions are needed.

The methodology followed for this project is not new (Peck, 1969;Bieniawski, 1984), but we decided to enforce the design framework philosophyas follows:

1. Evaluate risks, constraints, and site-specific conditions and conduct aknown-unknownreview to frame the requirements and potential mitigation measures.
2. Establish the dominant factors based on site conditions and establish a site investigation program that targets obtaining the nature, pattern, and properties of the materials.
3. Compile the information in such way that all sections are evaluated. Use plan views, cross-sections and profiles to form a three-dimensional view of the objective and prepare summaries that allow to establish most likely and worst-case properties.
4. Follow Peck's observational approach to design for the most likely case but identify the Key Performance Indicators (KPI) and mitigations for the worst-case scenario(s).
5. Use the KPI to establish an instrumentation and monitoring program in liaison with the contractor and the client to reduce the impacts to the construction planning and obtain the largest cost-benefit for the program.
6. Monitor and evaluate the dam performance continuously following the frequency required in the design. The data should be evaluated by the designer or by a qualified engineer familiar with the design risks, assumptions, and overall strategy.

We enforce the need to see the design as a process that goes from the conceptual phase to the detailed design phase and continues throughout the construction and operation and only ends with the closure of the designed facility. Design involves thinking, imaging, understanding the problem and providing solutions that, in this world, should be within a cost-effective framework.

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