From: Gamini Hettiarachchige [mailto:GHettiarachchige@ntpc.com]
Sent: May 11, 2015 11:23 AM
To: 'Nahum Lee'
Cc: Geraldine Byrne; Colin Steed; Jay Pickett; pewaschuk@wlwb.ca; Heather E. Beck
Subject: RE: N1L4-0150 and W2014L4-0001 Environmental Inspection Reports

Hi Nahum
Please find attached a copy of the paper presented at 2014 CDA Conference in Banff.
The Conclusion states that ***The installation of supplementary anchors recommended by other consultants was not necessary***.
Thanks

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From: Nahum Lee [mailto:Nahum_Lee@gov.nt.ca]
Sent: May-04-15 12:22 PM
To: Gamini Hettiarachchige
Cc: Geraldine Byrne; Colin Steed; Jay Pickett; pewaschuk@wlwb.ca; Heather E. Beck
Subject: RE: N1L4-0150 and W2014L4-0001 Environmental Inspection Reports

Gamini,

Thank you for your response to address the concerns raised in my inspection report, the documentation provided by NTPC confirms compliance with several of the outstanding recommendations in the 2011 Dam Safety Review. Once the Flood Design Study is complete in June 2015, I will expect updates from the NTPC regarding the outstanding recommendations for earthworks at Saddle Dam 1 and at the Cascades. Please keep me informed regarding the NTPC's plans to conduct restoration and maintenance work at the 5B Dam and Spillway as I would like to schedule my next inspection of the Snare Facilities to coincide with this work.

I have been in discussions with Board staff and we agree that despite the new memorandum from MECO and the additional report from Lloyd Courage, the rationale for the deviation from the 2011 DSR recommendation for post tensioned anchors seems inadequate. I am aware of a paper (although I have not read it), titled "Understanding the Physics: Analysis of Snare Rapids Spillway", written by Phil Helwig that, I understand, indicates there is no need for post tensioned anchors at the spillway. As discussed, this paper must be produced for the public registry as it will lend an additional degree of support to the rationale for not following MECO's 2011 Recommendation. The paper was presented at the 2014 CDA Conference in Banff.

Thank you for your attention and assistance. Take care and have a good day.

Regards,

Nahum

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UNDERSTANDING THE PHYSICS: ANALYSIS OF SNARE RAPIDS SPILLWAY

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ABSTRACT:

Snare Rapids Spillway (aka Spillway 5B) is unconventional in design. In recent years the stability of the structure has been revisited by three different consultants as part of dam safety reviews. These analyses came to markedly differing conclusions. In this paper we show that the approach which was based on a full three dimensional understanding the physics confirmed the safety of the structure. The other approaches which applied standard assumptions came to unnecessarily negative conclusions about the stability of the structure.

RÉSUMÉ:

L'évacuateur des crues (Snare Rapids - Spillway 5B) est d'une conception peu ordinaire. Ces dernières années, la stabilité a subit des analyses trois fois par de différents consultants dans le cadre d'études d'évaluation de sécurité des barrages. Les analyses arrivaient à des conclusions nettement différentes. En ce mémoire nous montrerons une approche, prenant en compte le physique de la situation, pour obtenir résultats plus exacts. Cette approche a d'ailleurs confirmé l'intégrité de la structure. Les autres approches basées sur les assomptions standards 2D donnent les résultats excessivement conservateurs.

1. INTRODUCTION:

Snare Rapids Spillway (aka Spillway 5B) is of unconventional design. It comprises eight bays each 20 feet (6.10 m) wide. Six of these bays are 8.3 feet (2.53 m) deep and two are 19.3 feet (5.88 m) deep, with respect to FSL. There are no rollways between the piers. The stoplogs seal against a low concrete weir, nominally 4 feet (1.22 m) high, except for Bays 3 and 4 where the sill beam is set at the design invert elevation. These features do not contribute any structural support to the piers. Stoplogs are handled by a mobile stoplog lifter with electrically powered hoists. The Spillway was constructed in 1960. Figure 1 shows a panoramic view of the spillway from downstream.

Figure 1: Snare Rapids – Spillway 5B: View from Downstream

Figure 2 (next page) provides dimensional details of the structure. Note that shear keys are provided for enhanced sliding resistance. The keys are 8.0 feet (2.44 m) long by 3 feet (0.91 m) deep. The piers are reinforced and dowels are provided as a further measure to enhance sliding resistance. The geology is typical for the Canadian Shield and the foundation rock is strong and massive.

The stability of this structure has been reviewed by different consultants in the course of three dam safety reviews giving contrary results. An analysis that properly represented the 3D behaviour of seepage loads on the structure found the structure to be stable without any need of additional strengthening.

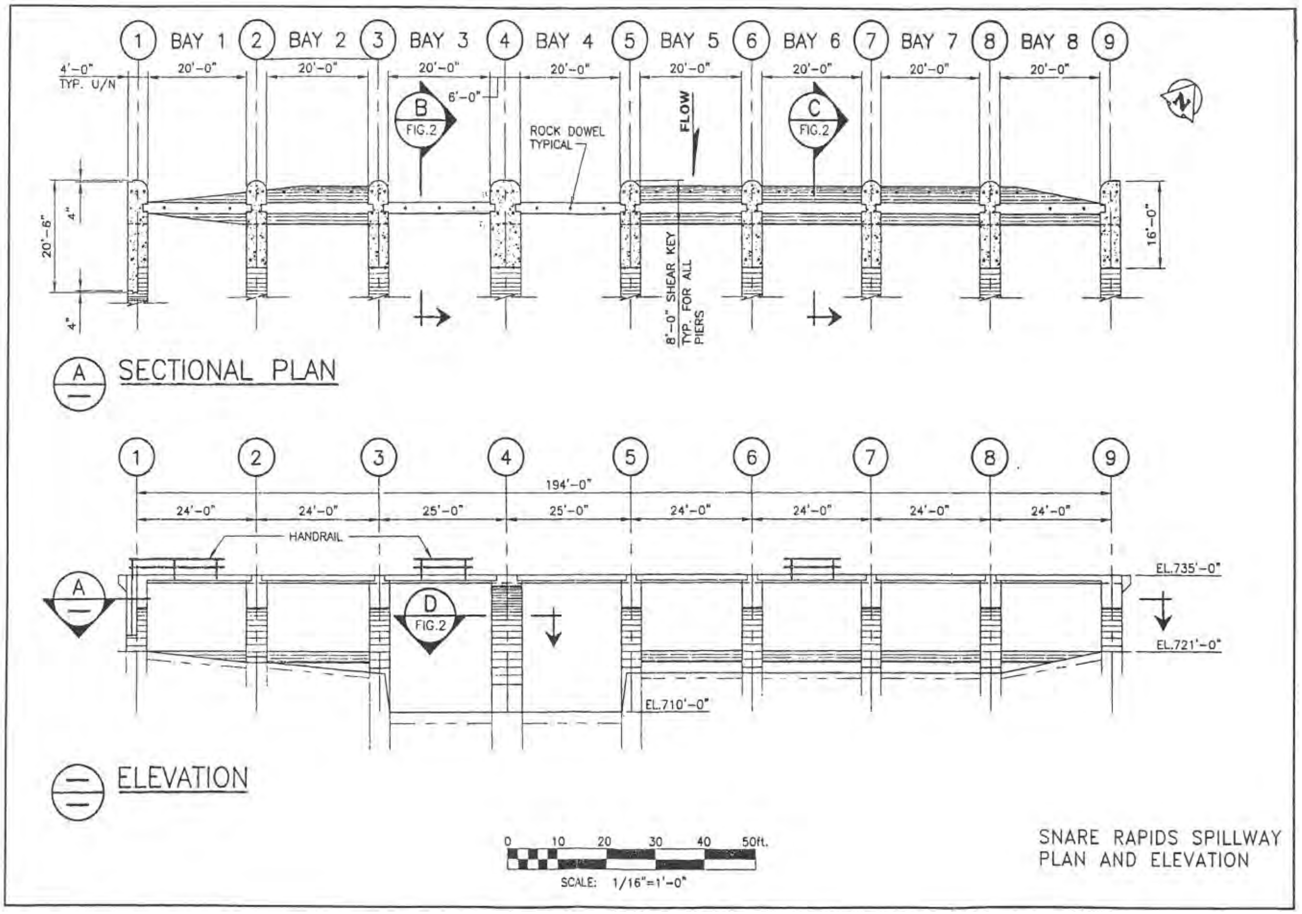

Figure 2: General Arrangements - Snare Rapids (5B) Spillway

2. STABILITY ANALYSIS

This review of the stability of the structure will focus on the critical case: Usual Load + Ice, for Pier 4.
The loads are calculated assuming the following parameters:

- HWL (FSL) = 222.29 m
- Sill elevation = 216.41 m
- TWL below sill
- Ice load on pier = 150 kN/m
- Ice load on stoplogs = 30 kN/m
- Ice loads applied at 0.3 m below water surface
- Water density = 1,000 kg/m^3 (9.81 kN/m^3)
- Concrete density = 2,400 kg/m^3 (23.5 kN/m^3)
- Uplift applied to 100% of footprint

Dimensional details are given in Figure 2 while the loading pattern is shown in Figure 3. The analysis is based on a plausible failure plane through the contact surface between dam and foundation, extending through the shear key (AA in Figure 2). Note the difference between the uplift loading pattern assuming 2D behaviour and the pattern based on 3D loading that recognises the physics of the situation. The recommended (3D) uplift pattern is based on an understanding of seepage flows along the assumed failure plane, taking into consideration that the structure footprint is narrow (104 m long x 1.83 m wide). Upstream of the stoplogs pressure on both sides of the pier will be equal and hence the uplift pressure would be buoyant hydrostatic. Downstream of the stoplogs seepage flow along the interface between concrete and rock can drain laterally and freely into the discharge channel. When actual seepage pressure and flow lines are considered the pattern will no longer be linear. The actual seepage pattern for this portion of the structure was determined by the relaxation method, following Harr (1962). The resulting loading pattern is shown in Figure3. This bears many similarities with the uplift design for Ambursen dams, which assume no uplift under the buttresses (Thomas, 1976).

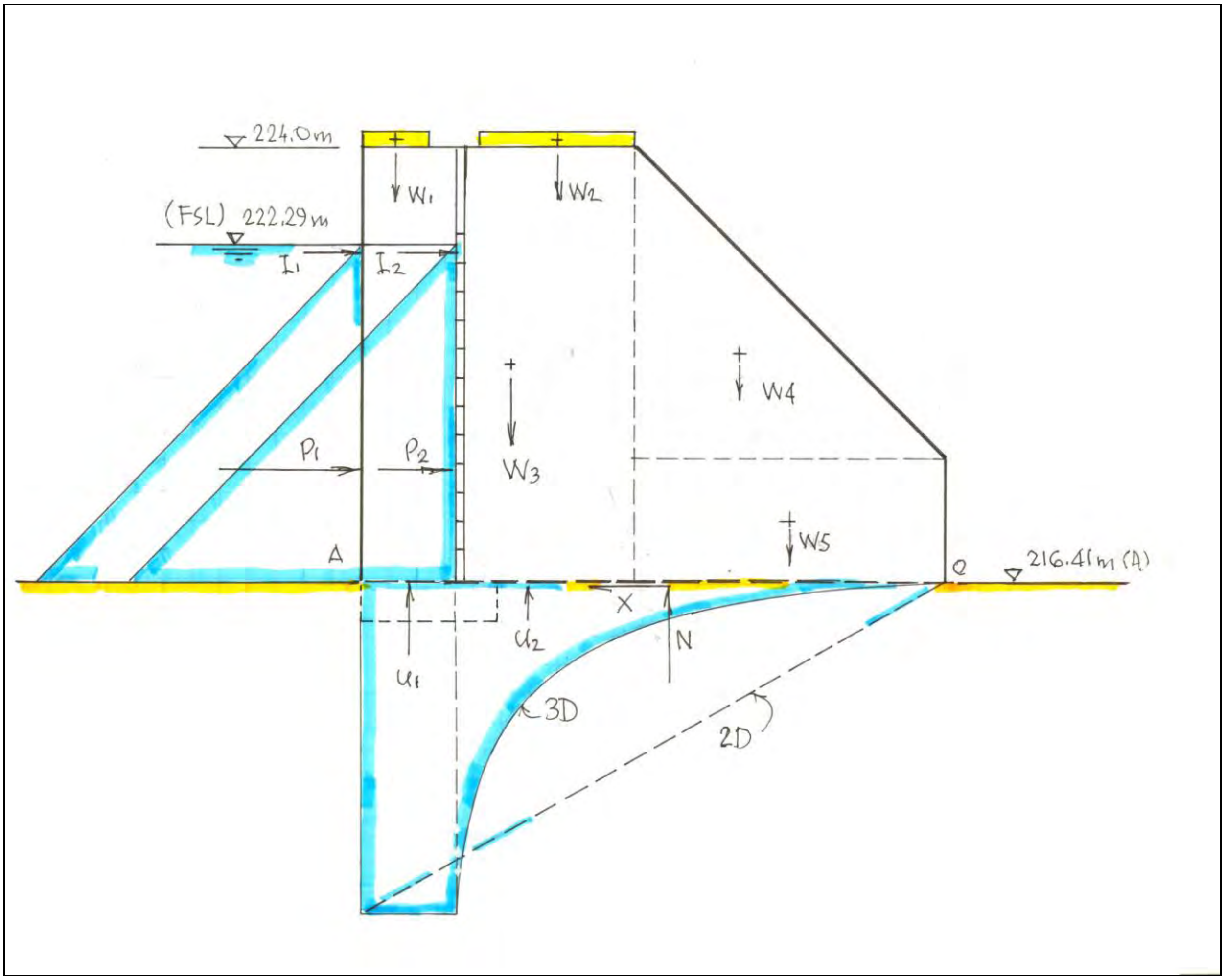

Figure 3: Load Pattern on Pier 4

The following acceptance criteria were applied:

- Position of resultant = within the middle third of the base
- Allowable concrete compression stress = 0.33f' (= 6,600 kPa for 20 MP concrete)
- Allowable concrete tension stress = nil
- Rock bearing stress assumed greater than concrete compressive strength
- Sliding factor of safety = 3.0 (Usual loading condition)

And material properties:

- Rock peak strength parameters: C = 1,300 kPa, tan $\phi = 55^{o}$; residual strength parameters: C = 0, tan $\phi = 38^{o}$
- Concrete peak strength parameters: C = 2,000 kPa, tan $\phi = 45^{o}$

Table 1 summarizes the results of the analysis.

TABLE 1: SUMMARY OF STABILITY CALCULATIONS

CASE 1: Usual + Ice

FORCE	COMPONENTS		ROTATION	LEVER	MOMENT
	HORIZ	VERTICAL		ARM	about O
W_1		68.3	1	9.14	624.3
W_2		153.5	1	6.25	959.4
W_3		1535.1	1	7.32	11,236.9
W_4		650.0	1	3.67	2,385.5
W_5		520.0	1	2.75	1,430.0
I_1	274.3		-1	5.58	-1,530.6
I_2	182.9		-1	5.58	-1,020.6
P_1	310.2		-1	1.96	-608.0
P_2	1034.5		-1	1.96	-2,027.6
U_1		-177.2	1	8.92	-1,580.6
U_2		-133.0	1	5.9	-784.7

ΣF_H (F)	1801.9					
ΣF_V (N)		2616.7				
ΣM_O					9,084.0	
$X = \Sigma M_O/N$					3.47	Within middle third.
					vs	
N/A	146.7	kPa			3.25	
Ne/Z	126.7	kPa				
f_{max}	273.5	kPa	< 6.7 MPa	OK		
f_{min}	20.0	kPa	> 0 kPa	OK		
Rock Properties (CEA, 1998)			Concrete Properties (BUREC, 1977)			
C =	1,300	kPa	C =	2,000		
ϕ =	55.0		ϕ =	45		
tanϕ =	1.4		tanϕ =	1.0		
SF =	16.6	> 3	OK	(Effects of shear reinforcement ignored)		
SF ~	16.1	> 3	OK	(Benefits of shear reinforcement included)		
C =	0.0	kPa				
ϕ =	38.0					
tanϕ =	0.8					
SF =	1.3	< 1.5	OK			(See discussion)

As shown in Table 1, the analysis concludes that Pier 4 is stable meeting all the acceptance criteria with ample factors of safety. Only the factor of safety assuming residual strength values for rock and concrete fails to meet the criteria. This result is considered to be spurious. This issue is further discussed in the next sub-section. The benefit of reinforcing in Pier 4 was also assessed assuming a typical reinforcement and calculating the shear benefit using a formula by Kono and Tanaka (2000). The results of this calculation were ambiguous but showed little benefit. This can be attributed to the fact that the assumed reinforcement ratio was minimal.

The second phase of the analysis was to examine whether the necessary shear capacity can be provided without failure in the foundation rock. Uncertainty about the condition of blast damaged rock was proposed by one of the consultants as a reason to reject the benefit of the shear key. He hypothesized that blast damage in the key way excavation would compromise the resistance of the bedrock. He envisaged a failure mechanism involving slippage of the rock wedge on the downstream edge of the shear key. Clearly there will be some blast damage to the rock surfaces resulting from construction of the shear trench. One would expect that all loose rock would have been removed and the key trench cleaned before concreting. Therefore conservative assumptions of rock strength properties would be in order. The failure condition would involve shear failure along the base and sides of the shear key as well as slippage along a failure wedge as shown in Figure 4.

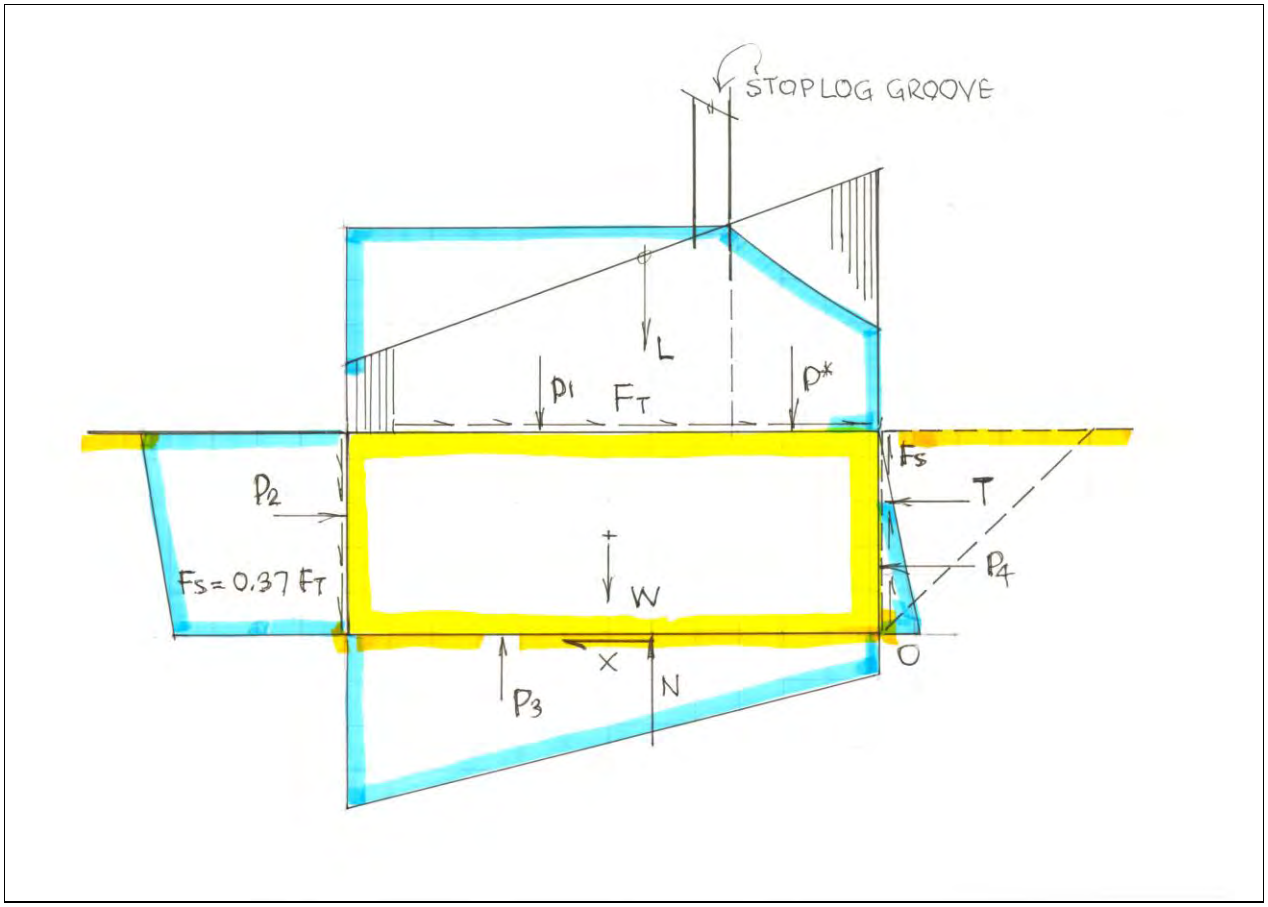

Figure 4: Shear Key Loads

It should be noted that the confinement or restraint of the key will facilitate mobilization of shear resistance and improve the inherent strength of the key. First the failure load (T) on the wedge was estimated. Two approaches were considered. The first hypothesis was that the rock was fragmented in the wedge zone and that a soil mechanics analysis assuming passive strength would be a reasonable and conservative. Taking $\phi = 42^o$ for calculating the longitudinal strength and lateral/longitudinal ratio = 0.3 gave a total strength of 927 kN. This strength value is primarily dependent on the overburden or counterweight stress from Pier 4. The second approach estimated the sliding strength of the wedge by a rock mechanics analysis assuming conservative values of $\phi = 42^o$ and C = 200 kPa. The failure slope was assumed to be 1V:2H. The resulting failure load was found to be 2206 kN. The second component of failure load is the sliding resistance of the shear key in its rock trench. The total resistance is the sum of these two components. This compares with the shearing load applied to the top of the shear key which is estimated to be 423 kN, assuming a proportionate distribution of shear force along the base of the pier. The stability of the shear key was assessed by a statics analysis treating the shear key as a free body. Details of the calculations and assumptions are summarized in Table 2.

Table 2: Summary of Stability Analysis of Shear Key

CASE 3: Shear Key

FORCE	COMPONENTS HORIZ	VERTICAL	ROTATION	LEVER ARM	MOMENT about O	REMARKS
F_S		157.7	1	2.44	384.79	$\tau_h = \tau_V = 94.7$ kPa
(-) F_S		-157.7	-1	0.00	0.00	
P*		46.5	1	0.3	14.0	
W		95.0	1	1.22	115.9	
P_1		184.8	1	1.57	289.2	
L_1		87.5	1	1.22	106.8	
L_2		119.2	1	0.81	96.9	
P_4	-17.8		-1	0.30	5.4	
$P_{2/1}$	90.4		-1	0.46	-41.1	
$P_{2/2}$	2.8		-1	0.61	-1.7	
$P_{3/1}$		-95.6	-1	-1.22	-116.6	
$P_{3/2}$		-73.5	-1	-1.63	-119.7	
F_T	422.8		-1	0.91	-384.7	
ΣF_H (F)	498.2					
ΣF_V (N)		364.0				
ΣM_O					349.0	
$X = \Sigma M_O/N$					0.96	
					vs	
N/A	81.6	kPa			0.81	(within middle third)
Ne/Z	52.2	kPa				
f_{max}	133.8	kPa	< 6.7 MPa	OK		
f_{min}	29.3	kPa	> 0 kPa	OK		
Rock Properties (peak):						
C =	600	kPa				
ϕ =	40.0	^{o}				
tanϕ =	0.8					
Sliding:						
SF =	13.2		> 3	With T =	927	kN
	11.3		> 3	With T =	0	kN
Rock Properties (residual friction)						
C =	0.0	kPa				
ϕ =	34.6	^{o}				
tanϕ =	0.7					
SF =	2.4	< 1.5		With T =	927	kN

The analysis showed ample factors of safety and that all acceptance criteria were met. Factors of safety were calculated with and without mobilization of the rock wedge. It is noted that even without support from the rock wedge that factors of safety are adequate. This is mainly attributed to resistance by cohesion which recognises 3D behaviour notably that cohesion is mobilized along both bottom and side surfaces of the key - double the resistance assuming cohesion to only act along the base! The analysis of the stability of the key-bedrock unit shows that the resistance against sliding provided by this design feature ensures safety against sliding failure with an ample factor of safety.

3. DISCUSSION

Unfortunately the original design approach and assumed rock and concrete properties are not known. Obviously the original designer was concerned about the resistance to sliding or overturning and decided to deal with the problem by incorporating a shear key into the structure. Rock dowels were also incorporated to provide enhanced resistance. The analysis done for this paper showed that most of the resistance to sliding came from cohesion and that anchoring by dowels was not required for stability. In many respects the behaviour of the shear key is analogous to the behaviour of tunnel plugs as discussed by Bergh-Christensen et al (2013).

A few points about design practice are discussed in this section. The first point is about the dual criteria that CDA recommends for calculation of factors of safety in sliding. As shown in Table 1 the factor of safety in sliding based on peak friction was 16.4 >> 3.0 required, whereas for residual friction the factor of safety was calculated to be 1.3 < 1.5 and thus fails to meet the criterion. This is not logical because residual friction can only be mobilized after loading beyond peak friction and involves some initial slippage and dilation along the critical surface of weakness. The seductive attraction of this method is that coefficients of friction (tanϕ) can be more reliably determined as there are fewer problems with scaling from sample size to prototype size. If both criteria are met it is wonderful but still not logical. Use of conservative safety factors as recommended by BUREC is to be preferred.

The second point arises from the assertion by one of the consultants that dam designs incorporating shear keys is not sanctioned under the norms of either FERC or CDA. The authors were unable to verify this statement. In researching this question a recent paper was discovered that discussed recent examples of the use of shear keys to reinforce foundation deficiencies in dams (Du, 1997). An observation from Du's paper is that the preferred location for a shear key would be toward the downstream face of the dam.

The use of dowels and pre-stressed anchors is a subject of some debate. CDA does not sanction use of anchors in design of new dams but admits anchoring to be a suitable solution for reinforcing existing dams that do not meet modern norms. Design of small concrete gravity dams subject to ice loads is a problem for the design engineer. Limited guidance on this subject is provided in available textbooks. However, it is interesting to note that the Norwegian code allows for use of dowels to enhance resistance to ice loads for small concrete gravity dams with heights of 7 m or less, Thomas-Lepine (2012). It is felt that the design approach should be decided based on engineering judgement, especially for small low hazard dams.

When the original designers of Spillway 5B developed their design they obviously assumed that steel dowels would be resistant to corrosion and have a service life comparable to other materials in the structure. Given that concrete is a protective medium for rebar one may assume similar benefits for dowels installed in concrete grout. Unfortunately there is limited data on long term performance. Work by Hassell (2008) suggests some approaches for assessing the risk of corrosion. Applying this approach seems to confirm that dowels a Spillway 5B should still be in good condition.

4. CONCLUSIONS

When faced with a problem that is unusual it is worthwhile to go "back-to-basics" and develop a hypothesis that accurately reflects the physics of the problem. For the case of Spillway 5B recognition of the 3D behaviour of seepage loads permitted a more exact analysis than alternative analyses assuming 2D behaviour. The 3D analysis showed that the structure was stable and thus the installation of supplementary anchors recommended by other consultants was not necessary.

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