Challenges in Assessing a Hydraulic Fill Dam Built in the 1920s for Liquefaction and Piping Potential

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ABSTRACT
The unsatisfactory performance of the hydraulic fill dams in seismically active areas is well known. Hydraulic fill placement technique usually results in loose fills that are susceptible to liquefaction under earthquake loading. Hydraulic fill placement technique was used during the original construction of a dam built in the 1920s in Alberta, Canada. This dam is partially founded on alluvium, which may also be prone to liquefaction. The earthfill embankments have zonation with core, transition zones and shells but without any filter zones or clear boundaries between the zones because of the hydraulic fill placement technique used during construction. The variable material composition and grain size from fine-grained to gravel make the assessment of liquefaction difficult as appropriate and multiple in-situ testing techniques and laboratory testing are required to assess their liquefaction potential. The absence of well-defined engineered filters and the variable grain size of the dam fills and foundation materials make the dam prone to internal erosion and piping also. This paper presents the challenges in assessing the liquefaction and piping potential of the hydraulic fill dam.

RÉSUMÉ
Les performances insatisfaisantes des barrages de remblai hydraulique dans les zones sismiquement actives sont bien connues. La technique de pose de remblais hydrauliques donne généralement lieu à des remplissages lâches susceptibles de se liquéfier sous l'effet d'un tremblement de terre. La technique de remplissage par remplissage hydraulique a été utilisée lors de la construction initiale d'un barrage construit dans les années 1920 en Alberta, au Canada. Ce barrage est partiellement fondé sur des alluvions, qui peuvent également être sujettes à la liquéfaction. Les remblais en terre ont une zonation avec noyau, zones de transition et coques, mais sans zones de filtrage ni limites claires entre les zones en raison de la technique de placement de fil hydraulique utilisée pendant la construction. La composition du matériau et la granulométrie variables du grain fin au gravier rendent l'évaluation de la liquéfaction difficile, le cas échéant. De nombreuses techniques de test et des tests de laboratoire sont nécessaires pour évaluer leur potentiel de liquéfaction. L'absence de filtres techniques bien définis et la granulométrie variable du remblai du barrage et des matériaux de fondation rendent le barrage vulnérable à l'érosion interne et aux conduites. Cet article présente les défis à relever pour évaluer le potentiel de liquéfaction et de tuyauterie du barrage de remplissage hydraulique

1 INTRODUCTION

The unsatisfactory performance of hydraulic fill dams in seismically active areas is well known. Hydraulic fill placement technique was used during the original construction of the earthfill embankments at a dam site in Alberta, Canada. This technique usually results in loose fills that are susceptible to liquefaction under earthquake loading. Even if liquefaction is not triggered, the soils may still develop high pore pressures and lead to large displacements affecting the seismic stability or post-earthquake performance of the dam.

The dams are partially founded on alluvium, which may also be subject to liquefaction and affect the seismic performance of the dam. The earthfill embankments have zonation with core, shells, transition zones but without any filter zones or clear boundaries between the zones due to the hydraulic fill placement technique used during construction. The materials within the core, shell and transition zones are highly variable and may be gap-graded at some locations. These material characteristics make the embankment prone to internal erosion and piping.

This paper presents the challenges in assessing the liquefaction and piping potential of a hydraulic fill embankment dam.

2 BACKGROUND

The 24 m high dam was constructed in the 1920s using the hydraulic fill placement technique. The dam is a zoned dam with a hydraulic fill core and transition zones and mechanically placed shells. The core was placed on bedrock while the shells were placed on the alluvium overlying the bedrock. Figure 1 shows the dam and zonation resulting from the hydraulic fill type construction and an idealized model of the dam.

The core gradation at the dam is variable and gap-graded. It comprises mainly silt with some sand and gravel. The transition zone is highly variable, ranging from materials that are primarily fine-grained, to sand and gravel with approximately 15% fines (silt and clay, finer than 0.075 mm).

The hydraulic placed material at the dam core and the transition zones could potentially be liquefiable depending on the relative density, grain size and the plasticity of the fines. The submerged granular materials within the shell and the foundation alluvium may also be prone to liquefaction. The liquefaction potential of these soils required investigation to assess the seismic stability of the dam. Standard penetration test (SPT) data for the core fill collected during an investigation in the mid-1980s had a wide range from very low values to high values. Values ranging between 15 and 25 were reported for the foundation alluvium. The higher values reported for the core and alluvium may be due to the presence of large particles within these materials and they will require careful consideration in the interpretation and the evaluation of liquefaction potential.

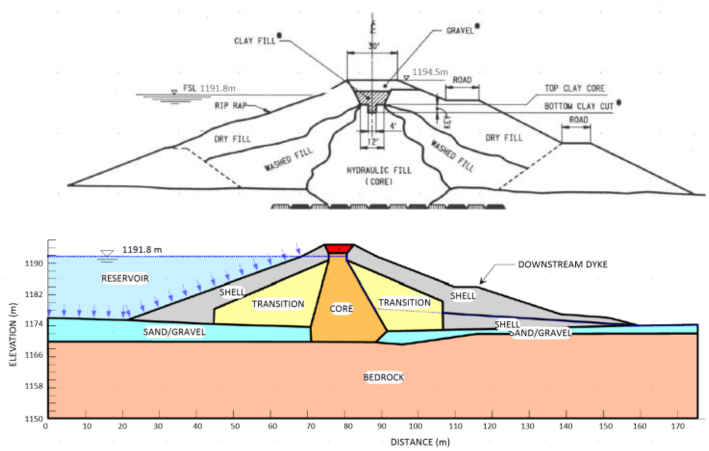

Figure 1. Dam, zonation and idealized model for the dam

Seismic instability due to liquefaction and internal erosion and piping were identified as key issues for the dam.

3 SEISMIC PERFORMANCE ASSESSMENT

The seismic performance assessment of the dam included the following:

- Site investigation and site characterization for seismic assessment;
- Seismic hazard assessment to develop ground motion spectra and time histories for dynamic analyses;
- Ground response analyses to assess the amplification of ground motions and to determine the earthquake induced cyclic shear stresses which may trigger liquefaction;
- Liquefaction assessment of potentially liquefiable dam fills and foundation soils; and
- Post-earthquake stability analyses using limit equilibrium methods to determine the post-earthquake factor of safety.

3.1 Site investigation for Seismic Assessment

The dam fills and the foundation soils at the dam consists of both coarse-grained (gravel and sand) and fine-grained (silt and clay) soil. Screening of liquefaction of coarse grained soils are typically performed using clean sand corrected SPT blow counts, $(N_1)_{60cs}$. In sandy soils, SPTs are performed. In gravelly soils, Becker Penetration Tests (BPT) are performed and the BPT blow counts are converted to equivalent SPT below counts for liquefaction assessment. The fine-grained soils are screened using their index properties (water content, Atterberg limits, grain size). If more detailed assessment is required for the fine-grained soils, undisturbed samples are extracted, and advanced laboratory testing is performed to assess their cyclic resistance using for example cyclic direct simple shear tests.

Liquefaction screening also require stiffness of both the coarse and fine-grained soils and is normally estimated from the shear wave velocities. Shear wave velocities are either directly measured using various geophysical techniques or estimated using the empirical correlations with the SPT data.

From historical investigation conducted in the mid-1980s at the dam site, SPT, BPT and Solid Cone (SC) and index testing data are available. However, the data were not very useful and may be unreliable for the following reasons: Conversion of BPT blow counts into the equivalent SPT blow counts would require bounce chamber pressure and/or energy measurements taken during the testing. However, they were not available. No reliable correlations are available to convert SC into equivalent SPT. For SPTs, energy measurements were not available. SPTs through the core were conducted in mud rotary drill holes. However, the SPTs performed outside the core were conducted inside the open Becker hole, which would render them as unreliable, since the Becker's air cyclone system would have softened the soils at the bottom of the hole. Also, for test locations below the water table, potential heaving might have softened the soils and made them unreliable. These two factors usually result in underestimation of blow counts or the relative density of the materials.

A limited scope site investigation program was undertaken to gather data and fill in data gaps. This investigation included: Two closed BPT holes with energy and bounce chamber pressure measurements, one open Becker hole with sampling and two tests pits.

Due to the coarse nature of the fill materials, BPT was selected as a suitable technique and the holes were advanced on the downstream bench. Typically, if the gravel content is more than 20%, the SPTs are considered unreliable. An AP1000 Becker rig fitted with an ICE 180 double-acting diesel hammer with a maximum rated energy of 11 kJ was used. 168 mm diameter (OD) double wall casing with an 8-tooth crowd out bit was used for both the closed and open Becker holes. Energy measurements were conducted using a pile driving analyzer (PDA) and an instrumented sub attached to the top of the BPT casings. The BPTs were converted to equivalent SPT $(N_1)_{60cs}$ using the Harder and Seed (1986) and Sy (1993) methods.

The interpreted Becker penetration test data is summarized in Figure 2. It also includes the laboratory index data. The equivalent $(N_1)_{60cs}$ interpreted from BPT data using the Sy and Campanella (1994) method were generally greater than the Harder and Seed (1986) method in the shell zone consisting of sand and gravel; however, the data within the transition zone consisting of sand and clay or clayey gravel agree with each other. Both methods predicted very high blow counts within the foundation sand and gravel unit.

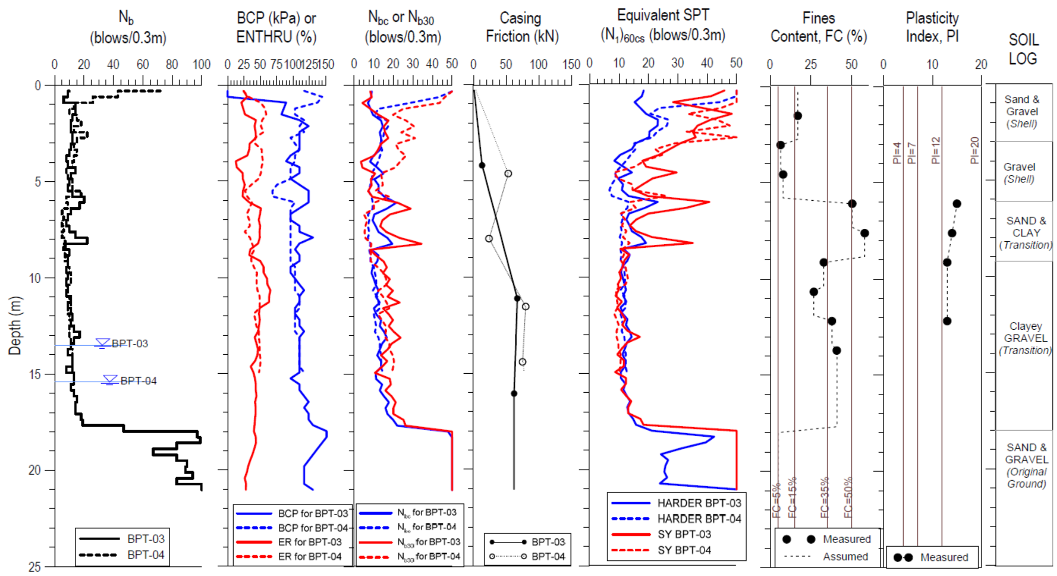

Figure 2. Becker penetration test (BPT) data interpretation summary

3.2 Geophysical Investigation for Seismic Assessment

Shear wave velocities and fundamental period of dams are key inputs in seismic assessments. An innovative and nondestructive geophysical measurement was conducted to determine shear wave velocities and the fundamental period of the dam.

Hammer impact Multi Channel Analysis of Surface Waves (MASW) test and ambient vibration tests were performed at three sites on the dam using linear arrays of Tromino seismic sensors. Additional ambient vibration tests were conducted along the crest of the dam. The fundamental period of the dam estimated from ambient vibration recordings varied between 0.25 second and 0.33 second (i.e. between 3 Hz and 4 Hz).

Figure 3 shows the shear wave velocity profiles at two locations on the dam obtained from geophysical survey. Shear wave velocities estimated from the blow counts data (SPT, SC and BPT) using empirical correlation between SPT blow count and shear wave velocities at three locations are also shown in Figure 3. Good agreement with the geophysical measurements at the two location is evident in Figure 3.

3.3 Seismic Hazard, Ground Motions and Time Histories

The dam is located at the foothills of the Rocky Mountains, close to the deformation front and west of the Canadian stable craton. A probabilistic seismic hazard assessment (PSHA) was conducted considering both the aleatory and epistemic uncertainties in the seismic hazard. A logic tree was used to handle the epistemic uncertainty included five source zone models for Canadian Cordillera, two source zone models for Canadian stable craton and three ground motion prediction equations (GMPEs). The estimated mean uniform hazard response spectra (UHRS) is shown in Figure 4. The Peak Ground Acceleration (PGA) corresponding to the 10,000 year return period UHRS is 0.19 g.

Seismic de-aggregation analyses, which provide magnitude-distance parameters for earthquakes contributing to the seismic hazard at the site, showed that the mean magnitudes representative for spectral period from 0 to 0.5 second is M6.1. The period range of interest for the dam is between 0.2 second and 0.4 second, which falls within this range. The corresponding modal magnitudes are M5.4 and M5.5. Thus, for the liquefaction and seismic displacement assessment, the representative magnitude is taken as M6.2 which is 0.1 unit greater than the maximum among the mean and modal values.

Time histories representative of the scenario earthquakes defined in terms of magnitude-distance pairs were selected from past historical earthquakes and uniformly scaled to approximately match the UHRS in the period range of interest for the dam. The earthquake intensity parameters for the selected and scaled time histories including Arias intensity and significant duration were also checked to ensure that they are representative for the scenario earthquakes as these parameters can affect the displacements from dynamic analyses (Thavaraj et al., 2018). A suite of ten scaled natural records are used in dynamic analyses.

3.4 Ground Response Assessment

Amplification of ground motions and liquefaction susceptibility of dam fills and foundation soils are the key issues affecting the seismic performance of the dam. One dimensional site response analyses were carried out using the computer program Proshake (Edupro, 2005) at five representative locations on the dam to determine peak horizontal accelerations and cyclic shear stress ratios (CSRs) within the soil column for liquefaction assessment. Figure 5 shows the location of the five soil columns. The ground motions were applied as “outcrop” motions at the base of the soil columns on the top of bedrock in the Proshake analysis.

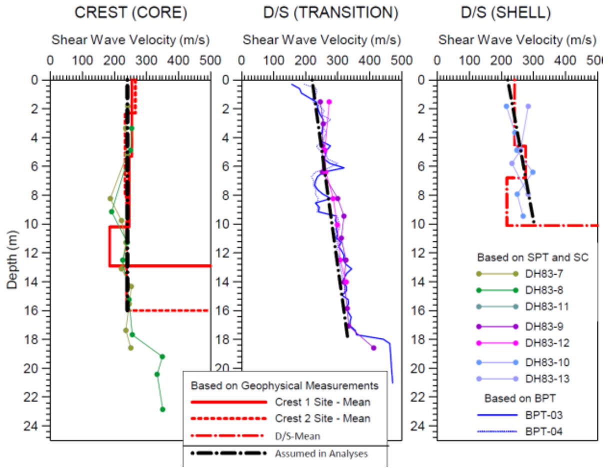

Figure 3. Measured and estimated shear wave velocity profiles

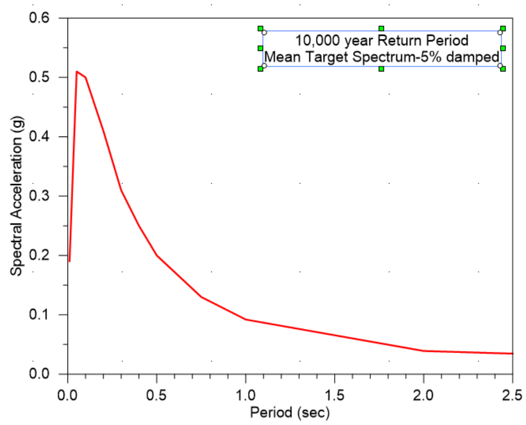

Figure 4. 10,000 year return period UHRS (PGA=0.19 g)

Figures 6 and 7 show the peak acceleration and CSR profiles at the five soil columns. The responses from the ten time histories fall within a reasonably narrow band in all the columns. Accelerations are generally amplified at the surface; however, the amplifications are not significant for columns through the crest (CR) and the upper part of downstream (DS1). Significant amplification occurs in the shorter columns DS2 and US2, and some amplification at US1. The magnitude of amplifications is slightly higher for the downstream columns than for the upstream columns. A similar trend was apparent the CSR profiles. CSR profiles show an increasing trend while moving from center either in the upstream or downstream direction. The CSRs at the upstream are comparatively greater than those at the downstream due to submerged conditions and decrease in effective vertical stress. Shorter soil columns both at the upstream and downstream show greater CSRs near the surface than the tall column at the crest apparently due to increase in the natural period of vibration.

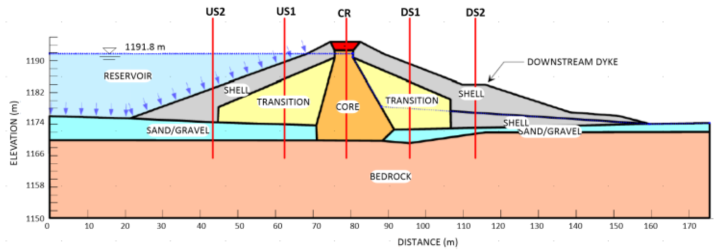

Figure 5. Idealized dam zonation and Proshake columns

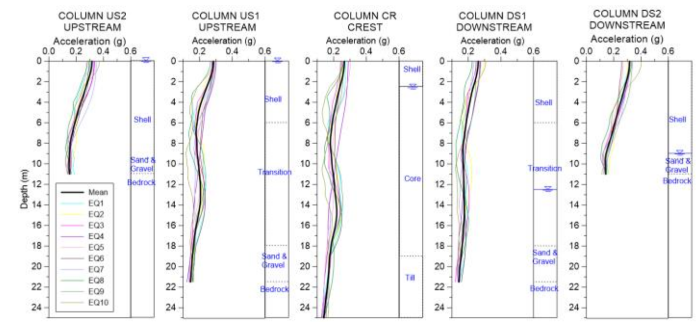

Figure 6. Peak acceleration profiles at the five soil columns

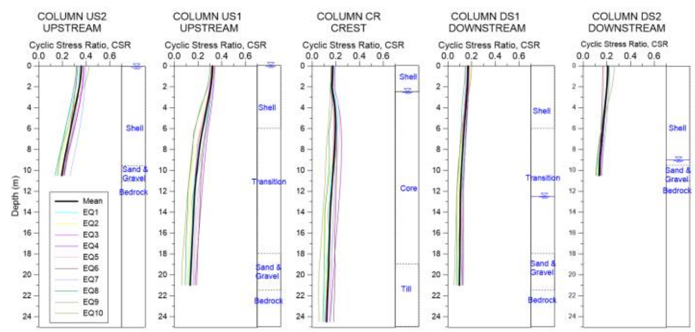

Figure 7. Cyclic stress ratio (CSR) profiles at the five soil columns

3.5 Liquefaction Assessment

The liquefaction assessment of granular soils was carried out in general accordance with the Seed simplified approach recommended by Idriss and Boulanger (2008). The cyclic shear stresses computed from Proshake and the SPT data were used in the liquefaction assessment. The liquefaction susceptibility of fine-grained soils was screened using the guidelines proposed by Idriss and Boulanger (2008) and Bray et al. (2004). Plasticity index (PI) along with liquid limit and water content were used as primary indices in screening for fine grained soils. If the PI<7, then the soils are assumed “sand-like” and potentially liquefiable and liquefaction is evaluated using Seed’s simplified approach. If the PI>12, then the soils are non-liquefiable and assumed to exhibit “clay-like” behavior with gradual strain development and without any significant strength loss except in the case of sensitive soils. If 7 ≤ PI ≤ 12, the material is considered less “sand-like” and less likely to liquefy with gradual strain development.

Figure 8 shows a summary of liquefaction assessment. At the downstream column DS1, liquefaction will not be triggered in the shell or transition zone under the downstream slope or in the foundation sand and gravel.

The fines content of the core is greater than 45% and the gravel content is less than 30%. Since the gravel content is not very high, the SPT blow counts from mud rotary drill holes through the core would not likely have been influenced by particle size and can be considered reliable. The fines are, however, low plastic with plasticity index of 8 or less and the liquefaction assessment treating it as “sand-like” show limited pockets of potential liquefaction where low plastic silt is present. Except for these pockets, most of the core will not trigger liquefaction.

No insitu SPT or BPT data was available for upstream soils. Figure 6 shows that the CSR induced by the earthquake upstream and the resulting $(N_1)_{60cs}$ required to prevent liquefaction at the upstream (US1) is generally greater than the corresponding column at the downstream side (DS1). If the relative density of upstream soils on the upstream side is same as that on the downstream side, it is likely that liquefaction will be triggered in the transition and shell zones on the upstream side. The assumption of similar relative densities both upstream and downstream is not unreasonable as the same hydraulic fill technique was used in the construction of the dam.

Note that at the downstream BPT holes (BPT-03 and BPT-04), within the transition zone, soils with very high fines content varying approximately between 30% and 60% with plasticity index (PI) greater than 10 were encountered as shown in Figure 2. These data suggest that the fine-grained soils with high fines content and PI > 10 may not show “sand-like” behavior. However, in the absence of any site specific data for the soils upstream, the transition zone at the upstream was conservatively assumed to exhibit “sand-like” behavior and expected to trigger liquefaction.

3.6 Post-earthquake Stability Assessment

Post-earthquake stability analyses were conducted using limit equilibrium method under post-liquefaction conditions to determine the flow slide potential of the upstream and downstream slopes. If the post-earthquake factor of safety (FOS) of the slopes is less than or equal to unity, flow slide or very large displacement will occur which may cause dam breach.

An analysis was conducted by assuming liquefaction would be triggered in the upstream transition zone and the downstream shell zone near the toe. Post-liquefaction residual strengths were assigned to these zones and the residual strengths were estimated using the correlation proposed by Idriss and Boulanger (2008). The resulting FOS for the upstream slope is less than unity (FOS=0.7) indicating that a flow slide type of failure will occur (See Figure 8).

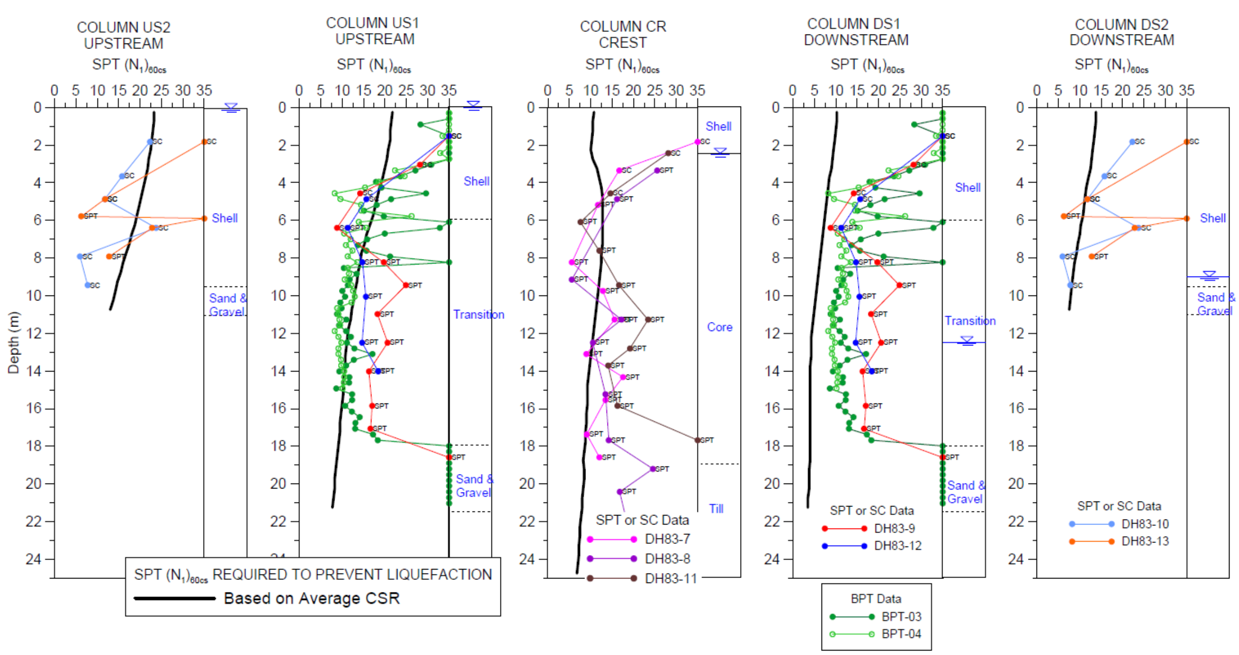

Figure 7. Liquefaction assessment at the five locations

On the downstream side, the FOS for the downstream slope is greater than unity indicating that flow slide type of failure of the downstream slope will not occur. However, the FOS for a slip surface confined to the downstream toe area simulating a localized failure through the liquefied sand and gravel is less than unity (0.8). However, this localized failure is not expected to extend to the dam crest and cause a loss of free board or dam breach.

In the absence of flow slide type failure, the embankments may still experience significant displacements under earthquake loading. Such displacements should not threaten the freeboard of the dam. A 2D or 3D finite element or finite difference based seismic deformation analysis using a constitutive model for soils, which can model the liquefaction process, is required to make accurate prediction of seismic displacements. In the absence of such detailed analyses and if liquefaction can be ruled out at key locations, simplified methods such as those based on the Newmark sliding block approach (Newmark, 1965, Bray and Travasarou, 2007) can be used to estimate permanent seismic displacements and empirical methods such as those by Swaisgood (1993) and Lo and Klohn (1992) can be used to estimate seismic settlements. Ignoring the potential flow slide type of failure of the upstream, the seismic settlement estimated using the empirical methods is less than 0.05 m. Therefore, if the liquefaction is not triggered in soils upstream, the dam will have adequate freeboard.

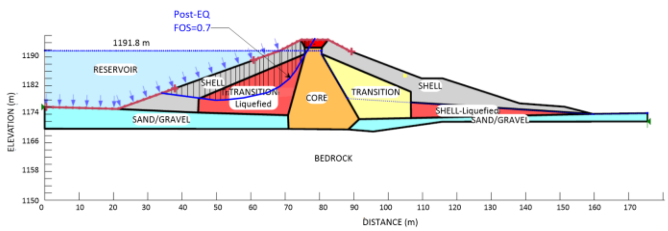

Figure 8. Post-earthquake stability of upstream slope

4 INTERNAL EROSION AND PIPING ASSESSMENT

4.1 Background and Data

Internal erosion and piping potential of the dam was assessed by computing the relative likelihood of failure by piping using the UNSW method proposed by Foster et al. (2000a,b). The filter compatibility requirements of the embankment zones were checked following the recommendations of FEMA (2011) and Sherard et al. (1984a,b).

Internal erosion and piping are a significant cause of failure and accidents affecting embankment dams. Three common modes of piping failures are due to piping: (1) thorough the embankment; (2) through the foundation; and (3) from embankment into the foundation. Analysis of failure statistics up to 1986 shows about half of dam failures are due to piping. About 46% failures occurred on first filling and another 24% within the first 5 years of operation. Only 2% occurred due to earthquakes.

Small concentrated leaks occasionally develop through the impervious sections of embankment dams. A satisfactory filter should not only be capable of sealing an initial concentrated leak, but also be stable during seepage with a high water pressure and a high hydraulic gradient.

For the hydraulic fill zoned dam partially founded on overburden soils, particle retention capability is key to assess the instability due to piping. During the site investigation for seismic assessment, data was also gathered to complement the available data from the mid-1980s to perform a detailed analysis of the particle retention capabilities of the embankment zones. Table 1 summarizes the grain size index properties of the embankment zones and the parameters used in the analysis of the particle retention capabilities: D_{15} and D_{85} (the grains size for which 15% and 85% of the soil, by mass, is finer).

4.2 Filter Compatibility and Particle Retention

As noted the 24 m high dam consists of an impervious core (silty clay with sand, low plasticity, firm to very stiff), transition zones (sandy clay with gravel, low plasticity, firm to very stiff) and shells (gravel and sand, some silt, compact to dense). The maximum pressure head at the base of the core is about 21 m. The core is founded on bedrock with a small part supported on a layer of sand and gravel. The transition zone overlies the sand and gravel layer and confines the core.

Table 1. Grain size parameters for various zones

Zone	Gravel	Sand	[a]Fines	D_{15} (mm)		[a]D_{85} (mm)	
	%	%	%	Range	[b]For Check	Range	[b]For Check
Core	0 - 31	13 - 21	48 - 85	< 0.015	0.015	0.1 - 0.2	0.15
Transition	12 - 47	26 - 38	27 - 58	< 0.01	0.01	0.6 - 1.7	1.2
Shell	39 - 67	27 - 44	6 - 17	0.06 - 0.87	0.40	2.7 - 3.4	3.0
Sand and Gravel	60 - 75	24 – 34	1 - 6	0.6 - 2.0	2.0	3.0	3.0

Note: [a] after re-grading for minus 4.75 mm; [b] values selected for check

Particle retention assessment was conducted considering all possible combinations of base and filter materials among core, transition, shell and the sand gravel using the FEMA (2011) guidelines. All combinations satisfied the acceptability criteria except for the sand and gravel. A sample in this material had maximum D_{15} of 2 mm, which did not satisfy FEMA (2011) requirement for protecting the core and the transition zone against internal erosion and piping. Following sections describe the review and assessment conducted to prove that, although the criteria is not satisfied, it is unlikely that initiation of piping will occur at the core and sand and gravel interface.

Sherard et al. (1984a) performed laboratory tests on sand filters with a pressure head of 40 m and high hydraulic gradients in the range of 1,000 to 2,000. The maximum D_{15} of 0.7 mm recommended in FEMA (2011) apparently corresponds to a lower bound value obtained in Sherard's study, and the D_{15} of 0.7 mm for the filter was obtained for a non-plastic silt with D_{85} of 0.039 mm. However, the D_{85} of the core and the transition zone at the dam are much coarser, ranging from 0.1 mm to 0.2 mm and from 0.6 mm to 1.7 mm, respectively. Review of Sherard (1984b) data suggest that, for D_{85} of the base soil between 0.1 mm and 0.2 mm, the required D_{15} of the filter ranges between 2.3 mm and 5.2 mm. For D_{85} of the base soil between 0.6 m and 1.7 mm, the required D_{15} of the filter varies between about 5 mm and 15 mm. Therefore, the sand and gravel layer with D_{15} of 2 mm is expected to have the necessary particle retention capacity.

A seepage finite element analysis was carried out with the computer program Seep/W (Geo-Slope, 2014), to estimate the pressure head, hydraulic gradient and flow velocity at the core – sand and gravel interface, which is the critical potential plane for initiation for the piping failure mechanism. Analyses were performed with and without concentrated leak that crosses the core and transition zone of the dam which were the initial conditions used in Sherard's experiments. The analysis indicated that, for both cases, the pressure head and hydraulic gradient at the core-sand and gravel interface in the dam are less than those used in the Sherard's experiments. The flow velocities are also 700-50,000 times slower than those in Sherard's study. The estimated low values of hydraulic gradient and flow velocity are apparently due to the low permeability of the sand and gravel layer, which was measured insitu as $7x10^{-5}$ m/s.

Thus, the hydraulic loading conditions on the core – sand and gravel interface of the dam which might be present are significantly lower than the ones used in Sherard's tests for determining the filter requirement D_{15}. This reinforces the conclusion that the sand and gravel has the necessary particle retention capability.

A sensitivity seepage analysis was also carried out to study the effect of the permeability of the core, the sand and gravel, and the shell on the hydraulic response. Two cases were analyzed by increasing the original permeability of the layers by 10 (Case 1) and 100 (Case 2) times. The results of the particle retention assessment are summarized in Table 2. The sensitivity analyses show very minor variations in the pressure head and in the hydraulic gradient at the core – sand and gravel interface as the permeability is increased by 10 and 100 times. The flow velocity increased, however, by about 60-200 times but still less than those measured in Sherard's experiments. Based on the analyses, it was assessed that the sand and gravel layer likely has the particle retention capabilities to prevent internal erosion and piping in the dam. This is consistent with the performance of the dam observed during more than 80 years of operation.

Table 2 Seepage analysis results and particle retention Assessment of the core zone

Case	Base Material		Filter Material		No Initial Concentrated Leak			Initial Concentrated Leak			Particle Retention Check Satisfied?
		$D_{85}B$ mm		$D_{15}F$ mm	H_p m	i	v m/s	H_p m	i	v m/s	
Sherard et al. (1984)	Silt and Clay with Sand	0.08-0.21	Sand-Gravel mixtures	2.3- 5.2	40	400	$> 5x10^{-6}$	40	400	$> 5x10^{-6}$	Yes
Case 1 (10*k)	Core	0.08-0.20	Sand and Gravel	0.6- 2.0	< 5	< 3.2	$< 4x10^{-8}$	< 21	< 3.5	$< 2x10^{-3}$	Yes
Case 2 (100*k)	Core	0.08-0.20	Sand and Gravel	0.6- 2.0	< 5	< 3.2	$< 4x10^{-7}$	< 21	< 3.5	$< 2x10^{-2}$	Yes

Note: H_p – pressure head; i – hydraulic gradient; and v – flow velocity

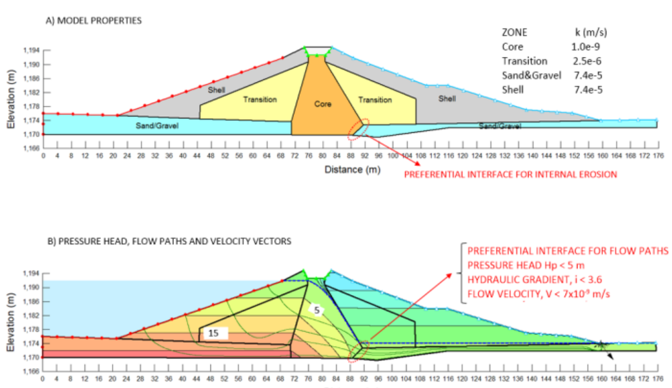

Figure 9. Seepage analysis without concentrated leak

5 LIKELIHOOD OF FAILURE BY PIPING

Foster et al. (2000b) developed a method called UNSW (University of New South Wales) method for estimating the relative likelihood of failure of embankment dams by piping. The three modes of piping considered in the UNSW method are piping through the embankment, piping through the foundation and piping through the embankment into the foundation. The method is based on analysis of historic failures and accidents in embankment dams. The underlying assumption in this method is that it is reasonable to estimate the relative likelihood of failure of an embankment dam by piping from the historic frequency of failures. The UNSW method is intended only for preliminary or screening level assessment.

In the UNSW method, the historic frequencies of failure by the three modes of piping are adjusted to consider the characteristics of the dam (core properties, compaction, foundation geology etc.) and the past performance of the dam. The adjustments are made with the use of weighting factors that are multiplied by the average historical frequencies of failures. Using the UNSW method, the annual likelihood of failure by piping, Pp, can be calculated using the following expression:

$$P_p = P_e \, w_{E(i)} + P_f \, w_{F(i)} + P_{ef} \, w_{EF(i)}$$

where P_e, P_f and P_{ef} are the average annual likelihoods of failure by piping in the embankment, in the foundation and of the embankment into the foundation, respectively, and $w_{E(i)}$, $w_{F(i)}$ and $w_{EF(i)}$ are the weighting factors that take into account of the dam characteristics such as the dam zoning, filters, age of the dam, core soil types, compaction, foundation geology, dam performance, monitoring and surveillance.

The estimated annual likelihood of failure by piping for the dam is $P_p = 4.1x10^{-6}$. If the assumed annual inspection of the dam is improved to monthly inspection, the annual likelihood of failure reduces to $2.5x10^{-6}$. An estimate of the probability of fatalities due to piping failure of the dam was computed assuming that the piping failure will result in loss of life; i.e. probability of failure by piping is equal to the probability of fatalities. The data falls within the ALARP (As Low As Reasonably Practicable) zone.

The intention of the simplified risk assessment is to provide a context for the likelihood of failure for the dam and not to conduct a formal and comprehensive assessment to assess risk with consideration of all possible modes of failure. The method used relies on the statistics of historical failures and judgement in the selection of weighting factors. Therefore, it is a qualitative indicator of potential for failure. More importantly, the method does not isolate and address the dams constructed using hydraulic fill method or with hydraulic fill against concrete faces at the interface between the earthen embankment and concrete dams.

6 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

The hydraulic fill placement technique used during original construction, data gaps, lack of adequate information from past investigations and the investigation techniques used in the 1980s made the assessment for liquefaction and piping potential challenging. A field investigation consisting of Becker Penetration Tests with energy and bounce chamber pressure measurements and test pitting were undertaken to improve the characterization of dam fills and foundation materials ranging in grain size from silt to gravel with cobbles. As the shear wave velocity and the natural periods of the embankment is important in assessing the seismic performance, nonintrusive geophysical techniques consisting of multichannel analysis of surface waves and ambient vibration tests were also used.

To assess seismic performance, several analyses including ground response analyses using time histories compatible with the uniform hazard response spectrum, liquefaction assessment and post-earthquake stability assessments were conducted.

The liquefaction assessment showed that the downstream slope will not liquefy and only limited zones with low plastic pockets of soils within core may liquefy. The seismic stability of downstream slope will not be undermined. There was no insitu data for the soils under the upstream slope. If the relative density of upstream soils is assumed similar to those at the downstream slope, the upstream shell and transition zone will liquefy, and the seismic stability of the upstream slope will be undermined. The primary reason for this conclusion is that the CSRs on the upstream side are much greater than those at the downstream side because of the submerged conditions. Site investigation with open and closed Becker drill holes through the upstream slope and laboratory index testing data are needed to assess and verify the liquefaction potential of the upstream soils.

Grain size and seepage analyses were carried out to assess the potential for internal erosion and piping of the impervious zones. The assessment indicates that the shells of the dams and the foundation soils have the required particle retention capabilities to seal a concentrated leak through the core and the transition zone and to prevent the initiation of a piping failure mechanism except as noted below.

The grain size analyses for the sand and gravel layer located in the foundation of the dam did not satisfy the particle retention requirements in FEMA (2011). However, a review of the Sherard et al. (1984) laboratory test data and seepage analyses show that internal erosion and piping will not be initiated.

The likelihood of failure by piping assessed using the UNSW method is 4.1×10^{-6}. A simplified risk assessment for loss of life resulting from the piping failure of the dam indicates that the risk falls into the ALARP zone (As Low As Reasonably Practicable) category.

The nature of hydraulic fill construction usually results in highly variable zonation with significant variation in gradation. These dams do not have compacted material with engineered filters as in modern dams. Therefore, there could be significant uncertainties in key properties affecting the erosion and piping potential such as the zonation, material gradation, and hydraulic gradients, differing from those determined based on available field or laboratory data. Therefore, it is prudent that any changes to dam geometry either due to excavation/filling or due to any other activity; intrusive investigation such as drilling; new installations for instruments or for any other purposes; unusual rate of rise or lowering of reservoir levels; and change in groundwater regime as observed in piezometric or seepage levels should be considered carefully. In addition, the effects of natural events (earthquake and flood) on erosion and piping potential should also need a careful assessment.

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