SEISMIC AND STRUCTURAL STABILITY ANALYSIS FOR INCREASED GROUND MOTIONS AT THE CORRA LINN DAM IN BRITISH COLUMBIA, CANADA

Katrina T. Wechselberger, P.Eng, Senior Engineer, Knight Piésold Ltd, Vancouver, BC, Canada.
Egbert J. Scherman, P.Eng, Specialist Engineer, Knight Piésold Ltd, Vancouver, BC, Canada.

ABSTRACT

Canada's extensive and aging dam infrastructure in combination with the updated and revised Dam Safety Regulations often require re-examining previously satisfactory analysis and developing a greater understanding of the cumulative and independent effects of a series of common design assumptions. Knight Piésold Ltd. recently completed a seismic and structural stability analysis on the Corra Linn Dam and Spillway located on the Kootenay River, approximately 15 km downstream of the city of Nelson, British Columbia.

Originally constructed in 1932, the dam and hydroelectric facility underwent repairs and upgrades as recent as 1990. The work consisted primarily of concrete re-surfacing, the installation of post-tensioned rock anchors to add the dam's stability, and the installation of piezometers to measure seepage under the dam and into the rock foundation. It was recommended in the latest Dam Safety Review (DSR) that the seismic stability of the dam structures be reassessed with the larger design earthquake values, in this case the Maximum Credible Earthquake (MCE).

Two separate 3D Finite Element Analysis (FEA) software programs were used to complete the analysis: Strand7 Software and ANSYS Simulation Software. This allowed for some confirmation of the results between programs and users. Pseudo-Static and Spectral Response analyses were performed. The various unknown factors such as actual material strengths, cracked structure/rock interface, etc. commonly encountered in a structure of advanced age can easily lead to compounded conservative assumptions. Where potential weaknesses are found it is important to understand what has contributed most significantly to the results so that recommendations can be well informed and clearly identify where further investigation is warranted.

RÉSUMÉ

Beaucoup d'ouvrage de retenue au Canada ont été construits il y a longtemps et compte tenu des révisions du règlement provincial sur la sécurité des barrages (*Dam Safety Regulations*), il n'est pas rare qu'il faille refaire des analyses, satisfaisantes à une époque, afin de mieux saisir les effets cumulatifs et indépendants d'hypothèses communes de conception. Knight Piésold Ltd. vient de réaliser une analyse de stabilité sismique et structurelle du barrage et de l'évacuateur de crue Corra Linn, sur la Kootenay, à une quinzaine de kilomètres en aval de Nelson (Colombie-Britannique).

Construit en 1932, le barrage a été réparé et révisé en 1990 : réfection de la surface de béton, installation d'ancrages post-contraint au rocher, afin d'accroître la stabilité du barrage, et installation de piézomètres pour mesurer l'infiltration sous le barrage et dans le substrat de roche. La dernière évaluation de sécurité du barrage a recommandé que la stabilité sismique du barrage soit réévaluée en prenant en compte les conditions de séisme maximal probable (SMP).

Deux programmes d'analyse par éléments finis 3D ont été mis à contribution : le logiciel Strand 7 et le logiciel de simulation ANSYS. Il a alors été possible de confirmer dans une certaine mesure les résultats obtenus par les deux programmes. Les résultats des analyses pseudo-statique et spectrale ont été évalués. Les diverses inconnues communes dans une structure construite il y a longtemps peuvent facilement mener à des hypothèses prudentes composées. Si des faiblesses potentielles sont découvertes, il importe de comprendre ce qui a contribué le plus largement aux résultats afin que les recommandations soient bien informées et permettent de dire à quel moment de plus amples études s'imposent.

1 INTRODUCTION

1.1 Facility Description and Study Background

The Corra Linn Hydroelectric Project, owned in part and operated by FortisBC, is located on the Kootenay River approximately 15 km downstream of the city of Nelson in southeastern BC, Canada. A mass concrete gravity structure constructed on bedrock, it is comprised of the following components:

- An east and west dam section, all regular concrete gravity sections
- A spillway concrete gravity section with gate slot piers and an ogee crest, and
- A powerhouse and associated headworks.

Originally constructed in 1932, the Corra Linn Dam underwent repairs as recent as 1990. The repairs consisted primarily of concrete re-surfacing, the installation of pre-tensioned rock anchors to add the dam's stability, and the installation of piezometers to measure seepage under the dam and into the rock foundation. The stability of the upgraded structures was assessed as part of a previous Dam Safety Review (DSR) completed in 2002 by others.

The need for further stability and structural analysis arose due the dam failure consequence classification being revised from "Very High" to a new "Extreme" category. The 2012 DSR addressed this revised classification, identifying that the annual exceedance probability (AEP) of the design earthquake had changed from 1/5000 to 1/10000 return period. As a result it was recommended in the 2012 DSR that the seismic stability of the dam structure be reassessed with the larger design earthquake values.

As is often the case with aging infrastructure, construction records do not lend themselves to easily defined material parameters and boundary conditions. For example, it was indicated on various historical reports and confirmed by the extensive network of rock anchors installed during the upgrades in the 1990s, that the Dam was constructed entirely on bedrock. No formal testing, however, has been completed to characterize the interface between the concrete structure and the bedrock. Furthermore the anchor heads were entirely grouted up, making it very difficult to re-tension the anchors or confirm their integrity.

A report was prepared that went through various iterations of sensitivity analyses to understand the effect of compounding assumptions and identify where more accuracy is warranted. This paper provides a discussion of the results and offers observations about the significance and effectiveness of various assumptions often relied upon in analyses such as these.

1.2 Review of Information and Existing Reports

1.2.1 Dam Safety Reviews

Dam Safety Reviews (DSR) for the Corra Linn Dam were completed in 1990, 2002 and most recently in 2012. The results of these reports helped inform the approach and the initial condition of the dam in this stability analysis.

Following the 1990 DSR, a series of upgrades were completed on the dam. Construction of this work was completed in stages and was ongoing from 1991 to 1996. A stability analysis of the dam was performed as part of the 2002 DSR to consider the upgrades of the previous decade. The dam was found to meet all the requirements according to the current Dam Safety Regulations. The upgrades were recorded in a set of as-built drawings and these upgrades were considered in the stability analysis performed for the purposes of this report as well. The dam upgrades and how they have been considered in this report are summarized in the following Table 1.

Table 1: 1990 DSR Upgrades & Stability Contribution

Upgrade Description	Stability Contribution in Analysis
Rock Anchors installed in all Gravity Dam Sections	1. A shear capacity of 60% of the Ultimate Tensile Load included in global sliding stability safety factors. 2. Load cases that include an initial vertical point load equal to 0%, 50% and 100% of the anchor's lock-off load included in the structural model used to assess the concrete capacity, the overturn base reaction and the sliding stability. 3. The results of the water tests performed during installation of the anchors are inconsistent and suggest that cohesion may only exist in some locations of the concrete to bedrock interface.
Concrete Rehabilitation	Structure's initial geometry & concrete quality preserved, supporting the use of the full dead weight of the structure as shown in the As-built drawings and average concrete material properties.
Installation of Piezometers	1. Some favourable readings from the piezometers over the years suggest that seepage under the dam has not increased. Some piezometers were recorded as dry suggesting there is little to no seepage in places. However, inconsistent results and maintenance practices do not provide justification to reduce uplift pressures resulting from seepage below the design status quo of full uplift at the heel and linearly reducing to zero at the toe. 2. Some readings suggest the foundation conditions have not deteriorated significantly and it is reasonable to assume some cohesion (bond) would exist between the concrete and founding rock at the onset of a seismic event in areas where water test performed during anchor installation did not fail the initial test. 3. Some questionable (potentially unfavourable) readings in localised places recently reported in December 2015, most notably in the middle section between the spillway and the powerhouse, suggest that uplift pressures could exceed the conservative assumption of full uplift at the heel, linearly reducing to zero at the toe. This may be a result of unreliable instrumentation and has not been included in this analysis. Further investigation and review of the piezometers is recommended to determine the uplift pressures that the Corra Linn Dam is actually exposed to.

1.2.2 Record Drawings

As-built Drawings (Record Drawings) of both the remedial works completed in the 1990s and some of the original 1932 construction records were provided by FortisBC. Information taken from these drawings that is pertinent to the stability analysis, as discussed in this paper, include:

- Locations and capacities of the rock anchors installed in the 1990s.
- Approximate depth to bedrock. For each section of the dam analysed, the deepest point was used to establish a worst case scenario. Specific rock elevations were reported on the Piezometer detail drawings. These were approximately consistent with the bedrock contours shown in plan.
- Section geometries as shown on various drawings.

1.3 Site Visit

A site visit of the Corra Linn Dam was conducted on October 2, 2015. The visit consisted of a visual inspection of the dam structure and a series of Schmitt hammer tests to investigate the compressive strength of the mass concrete in the Dam..

Figure 1: Corra Linn Dam Spillway during Site Visit

1.3.1 Site Visit Observations

As previously noted there were no indications of excessive seepage under the toe of the Dam. If there was seepage however, it would not necessarily be apparent as the toe of spillway is naturally wet due to moderate leakage from the spillway gates. The non-spillway portions have low levels of backfill over the toe. It was noted that most visible toe drains seems to be dry. Seepage was observed through some horizontal joints in the Dam's concrete in a number of locations. This seepage is visible on the downstream side of the dam despite 200 mm thick concrete resurfacing that was completed circa 1995. This suggests likely cracking on the upstream face of the dam, particularly at bed construction joints. In these locations, the concrete gravity structure is unlikely to have the same cohesive properties, as what one would expect from solid concrete and should be analysed as a smaller gravity structure with the potential to slide or overturn at this interface.

There was some deterioration of the concrete on the upstream noses of the spillway piers where the concrete surface are exposed to freeze/thaw weather cycles and are not submerged by the reservoir. This deterioration appeared largely superficial and is not expected to affect the stability or structural integrity of the dam.

1.3.2 Schmitt Hammer Tests

In the absence of destructive testing like drilling core samples out of the Dam, Schmitt Hammer testing estimates the strength of the concrete by a measure of the concrete hardness in a non-destructive way. During the site visit a number of Schmitt Hammer tests were performed at various locations on the Dam structure. All tests performed suggested that the concrete strength was adequate (as assumed in the stability analysis), even when the low outliers are considered in averages.

2 STRUCTURAL ANALYSIS

2.1 Loading

2.1.1 Reservoir Water Surface Elevations

The maximum normal operating water surface corresponds to the gates being closed. It also serves as the water surface for both the extreme earthquake and post-earthquake load cases.

Although the flood case was not a primary concern of this study, the extreme design flood reservoir levels were analysed to provided context and verify previously completed analyses. For the extreme design flood load condition, the hydrostatic water surface is applied to upstream side of the closed gates and to the piers so it does not represent a traditional flood condition where the gates would be open. As previously explained the PMF case was considered during the 2002 DSR and found to meet all the stability requirements. This assessment considers a couple alternate scenarios related to the Extreme Design Flood water levels and gates closed condition such as:

- The dam is spilling but considers the local stability if two of the fourteen gates are stuck in the closed position. The gates are aging and not automated so this load condition is not an inconceivable scenario.
- The gates are not opened in a timely matter and the water level is allowed to increases inside the reservoir to the crest of the gates without any water spilling or tailwater developing.

The water surface levels used have been taken from the Dam Breach Inundation Study completed in 2012 and are as follows:

- Maximum Normal Operating Reservoir Elevation: 531.876 msl.
- Maximum Design Flood Reservoir Elevation: 533.705 msl.

2.1.2 Extreme Design Earthquake

As required by the "Extreme" consequence classification assigned to the Corra Linn Dam, the design earthquake that should be used in the stability analysis is the Maximum Credible Earthquake (MCE), in accordance with the 2013 CDA Dam Safety Guidelines (DSG). The following Peak Ground Accelerations (PGA) were determined by others to locally characterize the MCE:

- Horizontal PGA: 0.236g.
- Vertical PGA: 0.142g.

These values were used in the pseudo-static load approximations for the hydrodynamic load increase (Westergaard's Equation) and for the structure's global inertia loads.

The pseudo-static method is considered to be a conservative method under the CDA Dam Safety Guidelines for establishing a dam's stability, especially in the case of large mass concrete dams where the global inertia loads are significant. Where results warranted a more detailed analysis due to either inadequate safety factors or nearing material failure criteria in the concrete section, a spectral response analysis was performed using the horizontal and vertical Uniform Hazard Response Spectra (UHRS) for an Annual Exceedance Probably of 1/10 000.

2.1.3 Silt Loading

There has been no reference to silt in the Corra Linn Reservoir in any of the previous the DSR's or other documentation provided to KP. The location of the dam, downstream of Kootney Lake, minimizes the opportunity for sediment transport to the reservoir. Furthermore the relatively shallow reservoir impounded by the Corra Linn dam and its tendency to be spilling, thereby maintaining increased flow velocities during a high flow event also minimizes the opportunity for sediment deposition inside the reservoir. It has therefore been assumed for all the primary load cases that there is no significant silt load acting on the upstream face of the reservoir.

One secondary load case has been included in the dynamic analysis with a silt level up to the Ogee Crest elevation to determine whether it is critical that a relatively silt-free reservoir be maintained. The saturated unit weight of the silt used for the purposes of the analysis was 20 kN/m^3 with an internal angle of friction of 30°.

2.1.4 Additional Loads & Assumptions

In addition to the above extreme events the following loads were considered in the analysis:

- Dead weight of gates in the spillway section of 36,700 kg each. It is assumed that the gates are in the closed position to have largest hydrostatic and hydrodynamic load transfer effect on the Dam structure.
- Full hydrostatic uplift reducing linearly to zero at the toe of the dam.
- Rock anchors installed and tensioned to the lock-off load reported on the As-built Drawings completed following the construction of the upgrades in the 1990s.

Excluded from the analysis (due to uncertainty or inconsistent records) are the following:

- Dead weight of the spillway superstructure, and other civil appurtenances such as walkways. These would only serve to add to the stability of the structure and do not represent a significant portion of the dam's self-weight.
- Any relief from uplift pressures one might otherwise consider as a result of favourable seepage conditions.
- As per Section 5.4 of the CDA Technical Bulletin: Structural Considerations for Dam Safety, it is not considered necessary to combine ice loads with the extreme earthquake event.

2.1.5 Load Case Combination Summary

Table 2 below represents a summary of the primary combined load cases as discussed above and the secondary load cases that were analysed to establish the sensitivity of the dam's performance, with and without the assistance of the rock anchors installed in the 1990s and a silt load that may or may not exist.

Table 2: Load Case Combination Summary

Primary Load Cases	Concrete Self Weight	Silt to the Ogee	Structure Inertia	Hydrostatic	Westergaard's Hydrodynamic	Uplift	Rock Anchor Tension	Gate Self Weight
1: Max Operating Water Level	1.0	-	-	1.0	-	1.0	1.0	1.0
2: Extreme Flood	1.0	-	-	1.0	-	1.0	1.0	1.0
3: Extreme Earthquake	1.0	-	1.0	1.0	1.0	1.0	1.0	1.0
4: Post-Earthquake	1.0	-	-	1.0	-	1.0	1.0	1.0
5: Seismic Spectral Response*	1.0	-	SR	1.0	-	1.0	1.0	1.0
Secondary Load Cases								
6: Extreme Earthquake Half Anchors	1.0	-	1.0	1.0	1.0	1.0	0.5	1.0
7: Extreme Earthquake No Anchors	1.0	-	1.0	1.0	1.0	1.0	-	1.0
8: Extreme Earthquake with Anchor & Silt	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
9: Post Earthquake No Anchors	1.0	-	1.0	1.0	1.0	1.0	-	1.0

*Load Case 5, the Spectral Response Analysis only performed where it was warranted by the natural period of the structure or by potential issues identified in the pseudo-static analysis.

2.2 *Analysis methodology*

2.2.1 Software

Two separate 3D Finite Element Analysis (FEA) software programs were used in the analysis of the Corra Linn Dam: Strand7 Software and ANSYS Simulation Software. This allowed for some confirmation of the results between programs and users. A representative section of each of the main components of the dam was constructed in AutoCAD Civil 3D using the geometries presented in the as-built drawings. These were then imported into the FEA programs to be modelled as 3D isometric solids.

2.2.2 Section Geometries

There are four main components to dam, the spillway and the east, west and middle gravity sections. As noted above, a representative section of each of the components to dam was analysed. The results section below, focusses on the spillway and the east and west gravity dam sections only. The middle section (buttressed by sections of the Powerhouse) has over 50% more concrete in the sections and it met all stability requirements; it is not discussed in this paper.

2.2.3 Material Properties

All sections of the dam, as modelled in both programs, were considered to be isometric concrete solids with the following material properties:

- Unit Mass: 2300 kg/m³
- Modulus of Elasticity: 27.5 GPa
- Poisson's Ratio: 0.2
- Internal Angle of Friction: 55°, and
- Internal Concrete Cohesion: 1000 kPa.

The dam's stiffness and behavior during the processing of the model is primarily governed by the first three material properties above. The internal friction angle and the cohesion are primarily used during the post processing computation of the Mohr Coulomb (MC) stresses. They represent approximately one third of the assumed concrete compressive strength of 30 MPa. In other words, the internal cohesion of the concrete was modelled as a separate property as it contributes to the structure's stiffness and as it contributes to the post-processing results to ensure excessive displacements from flexible concrete do not skew results. This depends on the software being used to perform the analysis and what inputs it requires/allows.

2.2.4 Boundary Conditions

As indicated on various historical reports and confirmed by the extensive network of rock anchors installed during the upgrades in the 1990s, the Corra Linn Dam was constructed entirely on bedrock. To KP's knowledge no formal testing has been completed to characterize the interface between the concrete structure and the bedrock. Observations made during the most recent DSR suggest that there is minimal seepage underneath the dam. It is therefore reasonable to believe that the foundation of the Corra Linn Dam is in fair condition. In photographic records and as observed during KP's site visit, there are no sign of erosion or deterioration of the bedrock foundation. The various dam sections have been modelled using the following representative boundary conditions at its foundation:

- The bottom surface of the concrete mass is laterally restrained in translation (i.e. sliding). The horizontal base reactions resulting from this restraint form the basis for the sliding safety factors.

- The bottom surface of the concrete mass (interface between bedrock and dam concrete) is supported vertically with a compression only support under normal operating, flood, pseudo-static seismic, and post-earthquake scenarios. This is a non-linear boundary condition (or statically indeterminate) that the FE program must iterate to find a solution. It offers a clean look at where the base of structure is reacting in tension and compression to satisfy the global overturning stability criteria. It also prevents the results from showing artificial tensile stresses on the upstream face where a fixed vertical restraint would attract undue load and without showing the structure's flexibility through the base.
- The spectral response analysis, which requires the computation of natural frequencies of the structure, can only be run with linear boundary conditions. For this case, the base was restrained vertically and horizontally from translation.

2.2.5 Pseudo Static Method vs. Spectral Response Analysis

The pseudo-static load method, also known as the seismic coefficient method, estimates the effects of the earthquake by applying static loads to the structure that represent in the structure's inertia and the hydrodynamic force of the reservoir. It assumes rigid displacement of the dam and is best suited to assessing the global stability of fairly rigid concrete gravity dams. It assumes that the movement of the rigid structure mimics the Peak Ground Acceleration (PGA). As recommended by the CDA in Section 7.6 of the Technical Bulletin: Structural Considerations for Dam Safety, it should be used as a screening tool and if it indicates that seismic loading could result in problems, more sophisticated methods of analysis should be used. A pseudo-static stability analysis was performed for all four representative sections of the dam using the 3D finite element analysis software programs described in Section 2.2.1.

The US Army Corps of Engineers notes (Ref. EM 1110-2-6050) that structures with low periods of vibration (<0.05 sec) such as mass gravity dams, exhibit spectral accelerations that is near the PGA. This is an indication that a structure is undergoing the same accelerations as the ground, as simplified in the pseudo-static method. In other words, it suggests the structure is not flexible to experience amplified accelerations and higher earthquake forces. A natural frequency analysis was performed on each of the representative sections of the dam to determine any anticipated amplification of the ground motions.

A Spectral Response Analysis considers the structure's flexible response to ground motion using a natural frequency analysis and Uniform Hazard Response Spectra (UHRS). It is most warranted in cases where structure's natural period falls between 0.06 secs and 5 secs. This is the range where structures typically see the worst amplified earthquake loads. It can also provide a reality check on element stresses where the pseudo-static analysis is showing potential issues.

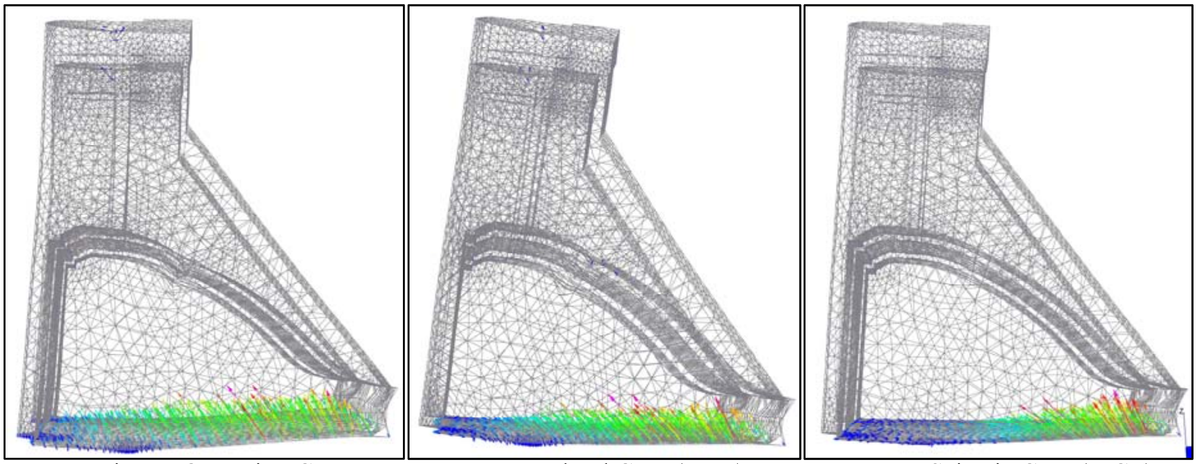
3 GLOBAL STABILITY

3.1 Overturn Resultant

The Dam's potential to overturn is characterised by the amount of the base that is in compression. When a significant part of the base is not in compression the structure is in danger of overturning. The CDA DSG requires that 100% of the base be in compression under the usual load conditions. Under the extreme earthquake load condition case, it is required that the base be partly in compression and that the compressive stress bearing on the founding material is lower than the allowable bearing stress of the foundation material.

3.1.1 Spillway Section

Figure 2 below shows the direction vectors of the nodal reactions for the Maximum Operating Water Level Case, the Extreme Flood - PMF Case and the Extreme Seismic - MCE - with the full anchor lock-off load included. It is clear from the vectors that the whole base is in compression for the Maximum Operating Case, while much less, approximately 55% of the base is in compression for the MCE Case.



Maximum Operating Case – 100% Compression | Extreme Flood Case (PMF) – 80% Compression | Extreme Seismic Case (MCE) – 55% Compression

Figure 2: Overturn Base Compression Reaction Vectors – Max Operating vs. PMF vs. MCE

A case was also analysed without the aid of the rock anchors installed in the 1990s and with the anchors providing only passive resistance (no pretension load). The MCE Case with the non-linear compression only boundary condition did not converge in the analysis when no anchors were present and it provided reduced base compression areas with passive resistance. This suggests that the spillway section of the dam may overturn during the MCE without the restraint of the rock anchors.

Of note, under the Maximum Operating Case the dam is stable for both the passive anchor condition (no pre-tensioned force in anchor) and the without-anchor condition, but neither condition satisfies the CDA criteria that requires 100% base compression as shown in Figure 3 below. Without the tensioned load from the anchors, all sections of the dam, including the east and west gravity dams should be considered non-conforming for the Maximum Operating Load Case.

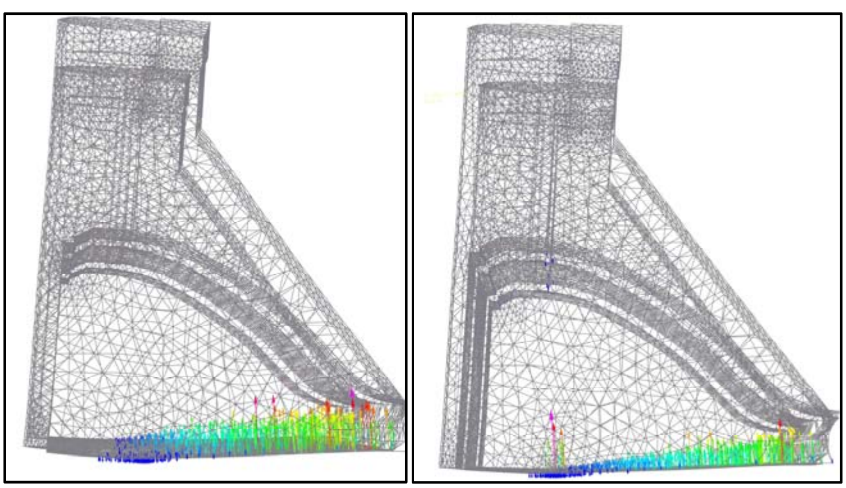


Figure 3: Maximum Operating Load Case with Two Compromised Anchor Scenarios Showing Less than 100% Compression

As there is no expected change to the hydrostatic or other live loads on structure, the Maximum Operating Case in Figures 2 and 3 also represents the post-earthquake condition. Any excessive cracking in the foundation bedrock during the earthquake may result in increased seepage and increased uplift conditions. As explained earlier in the report, full hydrostatic uplift load has been included in the analysis, decreasing linearly to zero at the toe. Uplift would never be greater than zero at the toe unless a static tailwater level develops. This condition (an increased tailwater level) is not expected to occur given wide channel downstream of the dam. The assumption to allow for full hydrostatic uplift allows for some foundation cracking and seepage to develop post-earthquake and is considered conservative for the normal operating case. It is however recommended that piezometer readings and visual observed seepage be monitored during a post-earthquake inspection to confirm no significant seepage and tailwater level develops which would increase the uplift forces on the dam beyond the design allowance, after an earthquake.

3.1.2 Non-Spillway Gravity Sections

All non-spillway gravity sections provided a satisfactory resultant in overturn with the anchors installed. Table 3 shows the approximate amount of the base that is in compression during the seismic event with and without anchors. Table 3 also includes the full results of the spillway, as described in Section 3.1.1 above.

Table 3: Overturn Base Compression

Section Location	Base Compression Percentage								
	Max Operating & Post Earthquake			Extreme Design Flood (PMF)			Extreme Design Earthquake (MCE)		
	With Anchors	50% Anchors	No Anchors	With Anchors	50% Anchors	No Anchors	With Anchors	50% Anchors	No Anchors
Spillway	100%	95%	75%	80%	65%	10%	55%	25%	0%
East Gravity Dam	100%	-	75%	65%	-	15%	60%	-	0%
West Gravity Dam	100%	-	90%	80%	-	55%	70%	-	0%

As with the spillway section, it is clear that the anchors contribute significantly to the overturn stability of the east and west gravity abutment sections of the dam.

3.1.3 Factor of Safety Against Sliding

A rigid body's sliding stability is characterised by the following formula:

$$\text{Safety Factor, } Q = C.A_c + (V.\text{Tan}\phi) / H \tag{1}$$

Where:

- C = Cohesion (bond between the concrete and the foundation rock)
- A_c = Base Area on in Compression
- V = Sum of Vertical Forces
- $\text{Tan}\phi$ = Coefficient of Friction between the Concrete Dam and Foundation Rock
- H = Sum Horizontal Forces

The CDA DSG notes in the acceptance criteria that given the significant impact a very small amount of cohesion can have on the shear resistance of small and medium sized gravity dams, safety factors that include cohesion should be used with extreme caution. It further recommends testing of the interface between the rock and concrete where possible. Where the bond is intact between the concrete and rock, cohesion values can be as much as half the tensile strength or roughly 1500 kPa for 30 MPa concrete). Safety Factors against sliding are presented below in Table 4 for two foundation conditions, one with just friction resistance (without any cohesion) and one friction resistance including 500 kPa of cohesion. The second scenario is meant to represent a reasonably bonded surface with some intermittent discontinuities or minor cracking/disbondment.

The CDA DSG does not stipulate a required Safety Factor during an extreme earthquake as it is not considered a catastrophic failure for a rigid body to displace since the direction of that ground motion causing the displacement is constantly changing. Given the 3D geometry of the Corra Linn Dam, with different sections extending in various directions connected to powerhouse, it is KP's opinion that a minimum Safety Factor of 1.0 should be achieved because the structure may not displace together in the same direction given unbalance horizontal load.

Included in Table 4 below are safety factors with and without anchors. It should be understood that that without the help of anchors, essentially represents a passive condition for the anchors. Without the lock-off load they could still restrain the dam in overturn but without the added normal force/base compression from the lock-off load, the friction generated from the increased normal force into the foundation is lost.

Table 4: Factors of Safety Against Sliding

Section Location	Safety Factor – With Anchors					
	Max Operating (& Post Earthquake)		Extreme Flood (PMF)		Extreme Earthquake (MCE)	
	Friction Only	500 kPa Cohesion	Friction Only	500 kPa Cohesion	Friction Only	500 kPa Cohesion
CDA Minimum FoS	1.5 (1.1)	3.0	1.1	1.3	Not Applicable (See Above)	
Spillway	1.8	6.4	1.5	5.0	0.9	1.5
East Gravity Dam	1.8	7.6	1.4	5.4	0.8	1.3
West Gravity Dam	2.1	10.9	1.4	6.5	1.0	2.5
Section Location	Safety Factor – No Anchors					
	Max Operating & Post Earthquake		Maximum Design Flood		Maximum Design* Earthquake	
	Friction Only	500 kPa Cohesion	Friction Only	500 kPa Cohesion	Friction Only	500 kPa Cohesion
Spillway	1.4	4.7	1.1	3.0	0.64	See Note *
East Gravity Dam	1.5	6.7	1.1	2.0	0.66	See Note *
West Gravity Dam	1.7	10.0	1.1	5.5	0.81	See Note *

*No value is provided for the cohesion case as the potential for overturn puts a large majority of the base into tension, negating the potential cohesion.

3.2 Concrete Capacity & Mohr Coulomb Failure Criterion

The concrete compressive stresses are within the limits required by the CDA DSG Acceptance Criteria for all loading scenarios and are not discussed in this paper. Besides the compressive strength of the concrete, it is important to understand the concrete's ability to withstand tensile stresses. Concrete is commonly understood to have a tensile strength of roughly 1/10 that of the compressive strength. Using the same acceptance criteria of 0.3 used for compression, a strength of 0.03 x f'c for tensile stress is used an indicator of where one might expect to see cracking and weakening of the concrete.

The most critical section with regards to the tensile strength of the concrete is the spillway, specifically the base of the pier columns on the spillway. Each pier has a rock anchor, adding an initial compressive load to the piers that helps reduce the local tensile stress on the pier column. Finite element analysis of a concrete body predicts failure most effectively based on the Mohr Coulomb Failure Criteria. The Mohr Coulomb (MC) Stress is a combined stress that considers the coordinated effect of shear and principal stresses in the concrete section. When concrete is in compression, it is capable of withstanding higher shear stresses. Similarly when concrete is in tension the amount of shear stress that it can withstand is reduced. A brittle failure is predicted when the MC stress is positive, or greater than zero.

Results show that the MC stresses are well below zero (safe zone) in all cases with the anchors installed. To understand how critical the pier anchors are, a case was run with the anchors in the ogee spillway to maintain stability, allowing the model to converge, but without the post tensioned anchors in the piers. In this case, the base of the piers neared failure during the MCE Case with perpendicular ground motion. Figure 4 below shows the extent of the MC stress at the base of the piers with and without the anchors installed. The red region indicates an MC stress that is nearing zero (failure). This result was further investigated in the dynamic spectral response analysis that is described in the sections that follow.

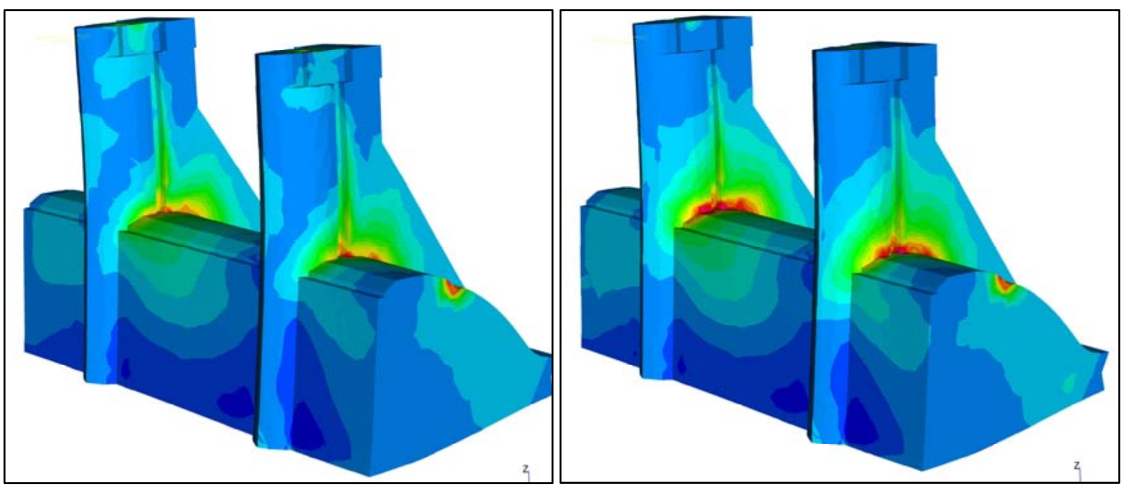


Figure 4: Longitudinal Ground Motion Mohr Coulomb Stress – With and Without Pier Anchors, Red Region Propagating When Compression in Piers from Post-Tensioned Anchors are not Considered

3.3 Dynamic Analysis

As previously explained in Section 2.2.4, a natural frequency analysis can indicate whether a dynamic analysis is required. A natural frequency analysis determines whether the structure is flexible enough to experience spectral accelerations that are amplified beyond the PGA.

3.3.1 Natural Frequency Analysis

The natural periods of each of the sections are presented in Table 5 below. Included for the spillway section is the period of vibration for ground motion parallel to the longitudinal and transverse axes of the Dam. This is relevant for the spillway section only due the flexibility of the piers in both directions.

Table 5: Natural Periods

Dam Section	Longest Mode Natural Period (sec)
Spillway – Transverse Ground Motion	0.045
Spillway – Longitudinal Ground Motion	0.068
East Gravity Section	0.040
West Gravity Section	0.030

The only Dam structure component with a natural period expected to be amplified by ground motion during a seismic (period greater than 0.05 sec) event would be the spillway section with longitudinal ground motion.

3.3.2 Spectral Response Analysis

Based on the pseudo-static results and the natural periods calculated from the natural frequency analysis, a spectral response (SR) analysis was performed on the spillway section only. The results of interest, based on what has already been discussed, are as follows:

- To determine whether the flexibility of the piers in their weak axis saw increased tensile stresses due the amplified spectral accelerations (longitudinal ground motion).
- To review sliding stability criteria based on nodal reactions to confirm how critical the anchors and bonded foundation conditions are (transverse ground motion).
- To include a static silt load up to the crest of the ogee for added sensitivity of the sliding stability of the spillway section.

A comparison of the tensile stresses from the pseudo-static analysis and the spectral response analysis is presented in Table 6 below:

Table 6: Spillway Piers Tensile Stresses & Displacements – Pseudo-Static vs. Spectral Response

Spillway Section	Maximum Operating & Post Earthquake		Extreme Design Earthquake (MCE)			
	With Pier Anchors	Without Pier Anchors	Transverse Ground Motion		Longitudinal Ground Motion	
			With Pier Anchors	Without Pier Anchors	With Pier Anchors	Without Pier Anchors
Tensile Stresses (MPa)						
Pseudo-Static	0.05	0.08	0.12	0.13	0.16	0.22
Spectral Response	NA	NA	0.10	0.10	0.33	0.38
Maximum Displacement at the Top of the Piers (mm)						
Pseudo-Static – Compression only Support	0.61	0.66	2.42	3.53	1.19	1.43
Pseudo-Static – Pinned Support	0.29	NA	0.65	NA	0.64	NA
Spectral Response – Pinned Support	NA	NA	0.48	0.53	0.80	0.84

The tensile stresses in the piers decreased with spectral response analysis when the ground motion was applied in the transverse direction. This is not unexpected for structures with low periods that behave rigidly and mimic the PGA, since the pseudo-static analysis has shown to be fairly conservative in such cases. When the ground motion was applied in the longitudinal direction, the tensile stresses at the base of the piers increased. Based on the 0.03 x f'c criteria previously outlined, concrete with a minimum compressive strength of 13 MPa would be required. This is anticipated to be well within the expected compressive concrete strengths of the Corra Linn Dam structure.

Reduced maximum displacement results are seen within the spectral response analysis, but it is important to understand that this is a reflection of the non-linear compression-only foundation support used in the pseudo-static analysis. In Table 6, the displacement results are included with the same foundation support condition that is used in the spectral response analysis. The results here show very minimal displacements if the foundation is held fixed (< 1mm displacement), but the minor changes to the results are consistent with effect on the tensile stresses. Transverse ground motion shows a reduction, while longitudinal ground motion shows an increase.

A comparison of how the base reactions from the spectral response analysis compare to the results from the pseudo-static analysis, are reflected by the Safety Factors against Sliding. In the analysis a silt load has also been included. The updated Safety Factors against Sliding are presented in Table 7 below.

Table7: Spillway - Safety Factor against Sliding: – Pseudo-Static vs. Spectral Response

Analysis Type	Extreme Design Earthquake (MCE) with Anchors	
	Friction Only	Friction with 500 kPa Cohesion
Pseudo-Static	0.9	1.5
Spectral Response	1.3	2.2
Spectral Response with Silt	1.2	2.0

As anticipated, the safety factors are improved when the SR analysis is used. The contribution of the anchors are required to maintain overturn stability but based on the results of the spectral response analysis, the presence of cohesion becomes less critical with a safety factor greater than 1.0.

The spectral response also allows for the addition of a silt load. Using the pseudo-static load case, the silt load would further emphasize the need for cohesion and the desired safety factors may not be achieved. As shown in the Table 7 above, based on the results from the spectral response analysis, the dam will be stable against sliding even when no cohesion is considered and with a silt load up to the ogee crest of the spillway.

3.4 Additional Sensitivity Analysis

3.4.1 Shallow Sections and Joint Seepage

As previously discussed, the presence of cohesion in the foundation increases the sliding stability of the dam dramatically. This is true for the stability of the dam at any horizontal joint (primarily construction joints) in the structure. Where a construction joint has deteriorated sufficiently in allowing seepage through the body of the dam, the dam should be analysed for the shallower section without cohesion/cementitious bond. Shorter sections represent both the dam above a compromised horizontal construction joint and sections of the dam where the bedrock profile is higher (i.e. reduced concrete gravity section). The previously noted joint seepage observed during the site visit confirmed the need for this assessment.

Shorter sections were analyzed for each area of the dam at the shallowest bedrock condition, and in the non-spillway sections at the horizontal joint where the slope of the downstream concrete face becomes steeper. The results for most sections were similar or better than the sections of full depth but were consistent in the need for the anchor tension loads with sliding safety factors below 1.0.

4 CONCLUSIONS AND RECOMMENDATIONS

Overall the Corra Linn Dam is expected to perform fairly well during the Design Earthquake (MCE) event if all stabilizing forces can be relied upon. Where there is uncertainty in the magnitude of some of the stabilizing forces and boundary conditions, a robust sensitivity analysis that includes both inclusion and exclusion of forces (as well as partial contributions) can provide some measure of comfort in the results of the analysis where major upgrades or decommissioning may otherwise be necessary / considered. The following is a summary of the key observations made during this study:

- Where foundation anchors are considered critical to the structural integrity of a facility and not used only to increase safety factors, means and methods should be provided to ensure the anchors can be pull tested or re-tensioned in the future.
- Cohesion, as a bonded surface between the concrete and rock, can contribute significantly to the stability of a dam structure. A sensitivity analysis that allows only a small amount of cohesion as opposed to a black and white approach of fully bonded condition (or not) should be considered.
- Similarly where an anchor's post tensioning cannot be relied upon, consideration can still be given to the "passive" support an anchor can provide – particularly as it relates to sliding stability.
- Pseudo-static results are conservative for most mass concrete structures where the natural period is low. Where ground motion amplification is not expected, a spectral response analysis can provide more refined results (and added safety factor).

5 REFERENCES

Canadian Dam Association (CDA). 2007. *Technical Bulletin: Structural Considerations for Dam Safety*.

US Army Corps of Engineers (USACE). 1999. *Engineering Manual 1110-2-6050: Response Spectra and Seismic Analysis for Concrete Hydraulic Structures*.