A bespoke analytical methodology for seismic safety assessment of spillway gates: case study of Waipapa dam spillway gates on the Waikato River

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ABSTRACT

This paper presents a seismic assessment methodology tailor-made for the Waipapa Dam Spillway gates on the Waikato River which were found to have potential dam safety issues under the 1 in 2500-year annual exceedance probability (AEP) earthquake actions. The seismic performance assessment of Radial Spillway gates was conducted under the combined actions of hydrostatic thrust and hydrodynamically induced loads. A 3D finite element PLAXIS model was developed and analysed with a selected ground motion suite to extract seismic demands at the base of the spillway and at the gate trunnions. An improved and bespoke methodology was developed to incorporate the fluid-gate interaction for hydrodynamic loads. Commercially available SAP2000 software was used to model the upstream reservoir using 3D solid elements with 'fluid-like' material properties and appropriate boundary conditions. The explicit modelling of the reservoir allows the fluid to interact with the spillway-gate assembly at their respective frequencies and includes auto-generation of hydrostatic loads. This approach is compared with the commonly used Westergaard Method using 'added masses' which assumes a rigid dam and incompressible fluid. Results shows good correlation between the two approaches and the benefit of reducing conservatism in the 'added mass' approach assumption of the hydrodynamic component always acting in-phase with the gate. The seismic evaluation found the gate components have sufficient capacity to withstand the demands during 1 in 2500 AEP event.

1 INTRODUCTION

1.1 Background

The Waipapa Power Station is owned and operated by Mercury NZ Ltd and is located on the Waikato River approximately 133 km downstream from Lake Taupo. The station was designed and constructed by the Ministry of Works commencing in 1955 and was commissioned in 1961.

Potential dam safety issues were identified in 2011 in the Comprehensive Dam Safety Report of Waipapa dam spillway. Further 2D analysis of the gate elements carried out earlier past revealed that under the combined actions of hydrostatic thrust and the hydrodynamically induced loads the gate's main horizontal and vertical girders are loaded beyond their capacities under 1 in 2500 year return period seismic event.

This case study focusses on evaluating the effects of 1/2500 annual exceedance probability (AEP) earthquake actions on the performance of the gates, and to propose designs of strengthening works if required, to provide a high degree of confidence on gate operability following the Safety Evaluation Earthquake (SEE) with an estimated 1/2500 AEP.

1.2 Hydrodynamic Interaction – Industry Practise

A common method to incorporate fluid-gate interaction in the analytical model of spillway gates is through 'Added Mass' method to account for the hydrodynamic loads. This approach is based on the work first published by (Westergaard 1931) and followed with modifications and improvements by others including (Salomon 2015). It is based on the assumption of rigid dam and gate and the reservoir is considered an incompressible fluid. These assumptions allow for simplification in the modelling of fluid-gate interaction by the use of joint masses to account for the lateral thrust of the reservoir fluid, usually termed 'Added Masses', instead of the finite element modelling of the fluid elements. These 'Added Masses' are applied on the skin-plate of the gate, and they oscillate with the same frequency as the gate plus the added mass to approximate the dynamic interaction of the gate with the reservoir.

The added mass function depends upon the shape of the vibration mode considered, so no one function will be exactly valid for all vibration modes of the structure. However, it has been shown that the added mass representing the interaction of a rigid structure can be used to account for the hydrodynamic effects for all modes of the non-rigid structure to an acceptable degree of accuracy for most purposes (Goyal and Chopra 1989) and (Davey et al. 2007). The following added mass function (Westergaard 1931) of earthquake forces on dams impounding water, is commonly used for the analysis of gates:

$$m_a(z) = \frac{7}{8} p_w \sqrt{Hz} \qquad (1)$$

where $m_a(z)$ = added mass at depth z below the surface, p_w = mass density of water, H = total water depth.

1.3 Improved Modelling for Hydrodynamic Interaction

An improved alternative to incorporate fluid-gate interaction in the analytical model of spillway gates is employed in this case study by using commercially available structural analysis software SAP2000. It utilizes the 3D solid elements to model the spillway block and the reservoir along with the gate. The 3D solid elements constituting the reservoir are provided with the 'fluid-like' material properties and appropriate boundary conditions at the side and the upstream boundaries and are connected to the gate skin by means of rigid links. The explicit modelling of the reservoir using 3D solid elements allows the reservoir to interact with the spillway-gate assembly with their respective modal frequencies and the lateral hydraulic pressures to be calculated automatically from the dead weight of the modelled reservoir (USBR 2006) and (Muhamad Aslam et al 2002).

2 WAIPAPA SPILLWAY AND GATE STRUCTURE

The spillway and diversion sluice structure is 32 m wide across the channel and 23 m deep and discharges in to the 32 m wide stilling basin. Each spillway opening is 7.5 m wide with a total spillway width including the side and middle walls of approximately 20.5 meters (Figure 1).

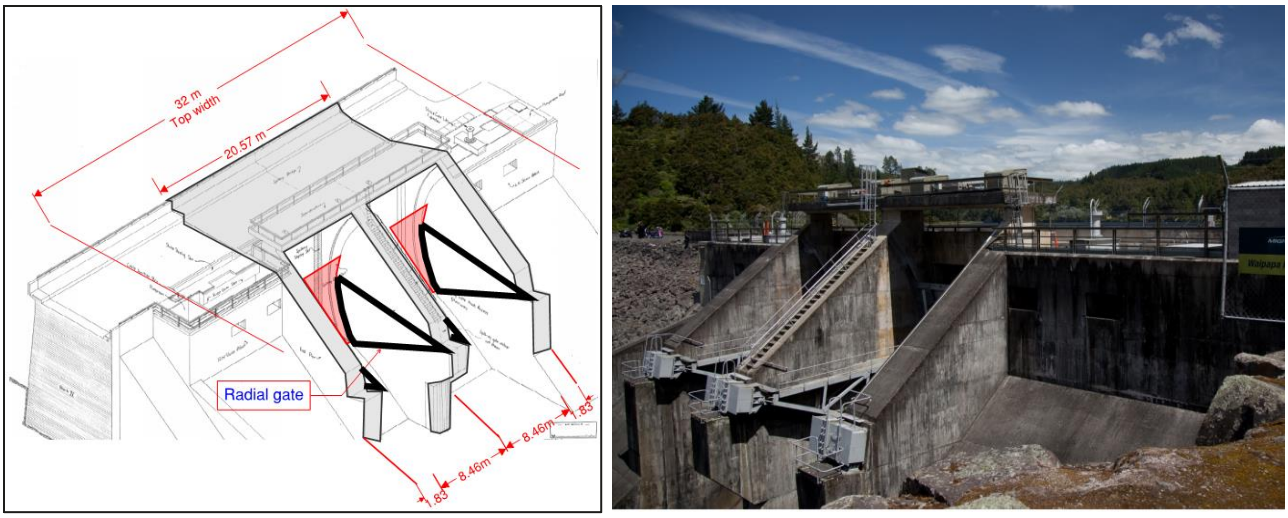


Figure 1 : Sluice and Spillway Structure - Waipapa Power Station

The spillway block has two identical 7.5 m wide,9.3 m high and 10.7 meters radius gates (Figure 2). The gates are sill-supported when closed and are operated by two wire rope winches connected to the bottom of the skin plate through lifting lugs welded to the vertical rib and skin plate assembly. The gate arms are connected to the trunnion pin housed in the trunnion girder which is anchored into the concrete spillway piers by means of steel rod anchor bars. The arms support the main top and bottom horizontal girders which in turn support the three vertical girders. The two end vertical girders are supported at an offset from the support of the horizontal girder causing load transfer through shear.

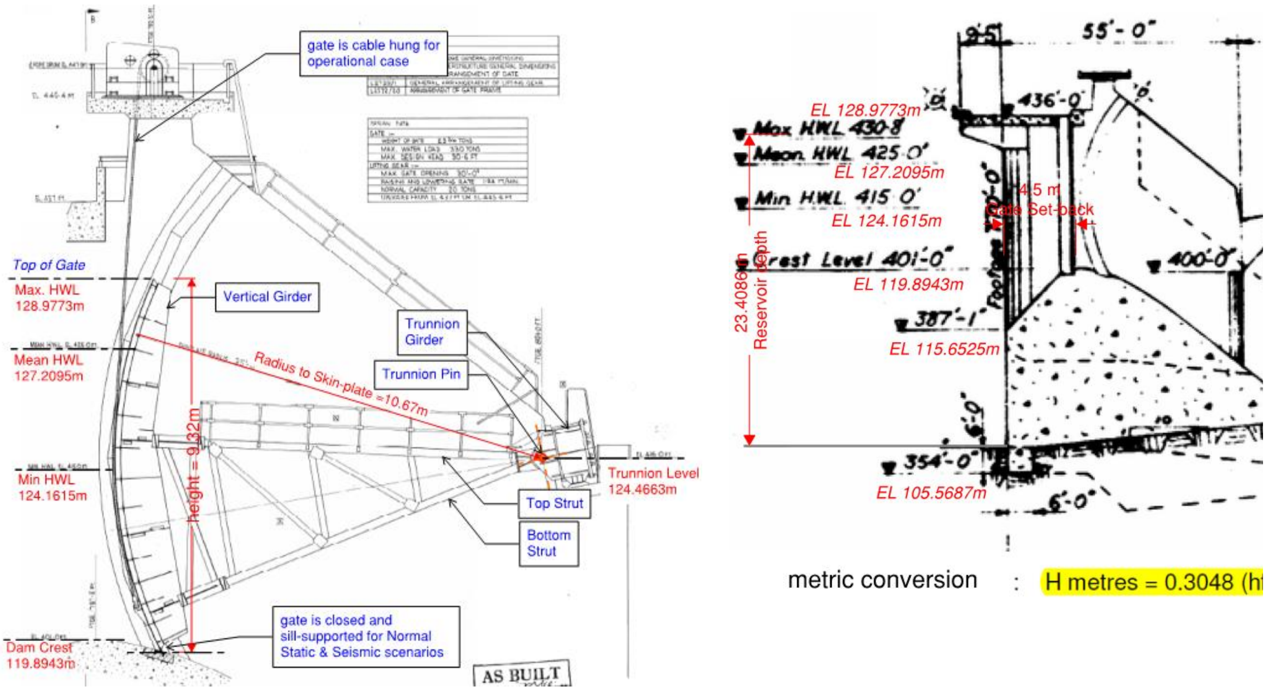


Figure 2 : Sluice Gate and Spillway Sections - Waipapa Power Station (from Original Drawings)

3 METHODOLOGY FOR ANALYSIS AND SEISMIC EVALUATION

The proposed analytical approach for gate assessment comprises a two-stage process to account for the following effects;

- Interaction of the spillway block with the underlying geology and its impact on the ground motion characteristics at the ground, the spillway base and at the gate trunnions.
- Interaction of the reservoir and the spillway-gate assembly to account for the hydrostatic and hydrodynamic loads

3.1 Stage 1 – 3D Finite Element Modelling and Analysis of Spillway and Ground

- Select ten (10) ground motion records and scale to 1 in 2500 year RP (SEE) spectra from the PSHA report as shown in Figure 3.

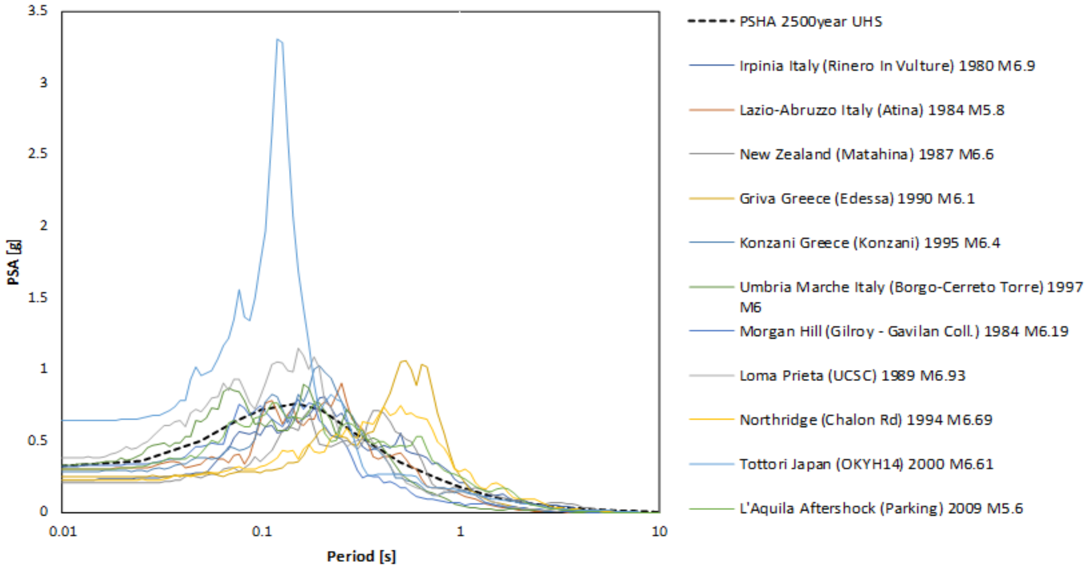


Figure 3 : Scaled spectral accelerations vs target spectra (1in 2500year RP), horizontal component

- Generate FE model of the spillway block and the surrounding ground to incorporate the amplification of applied ground motion histories from surface to the trunnion due to dam-foundation interaction.
- Determine acceleration and displacement time histories at the gate trunnion pins and the dam base.

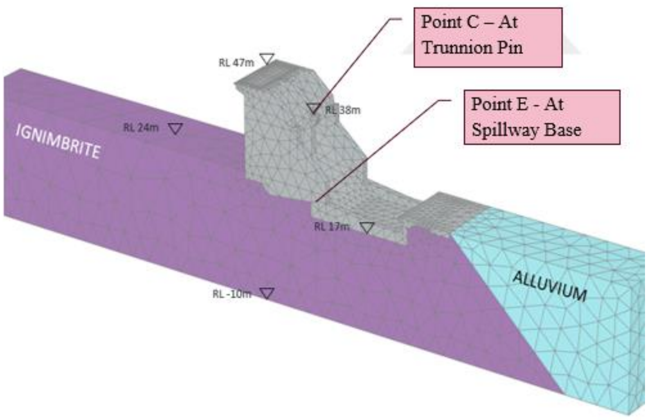


Figure 4 : Analytical model of spillway and ground

The ground motion records were applied to the 3D finite element model of the spillway block and the ground shown in Figure 4 developed in PLAXIS 3D. The interaction between the spillway block and the underlying rock typically resulted in amplification of response at the gate trunnions relative to the spillway base between the period range of 0.01-1 seconds whereas the amplification is insignificant for periods longer than 1.0 seconds.

The resulting time histories and their response spectra at the base of spillway block (Point E) and the trunnion pin (Point C) respectively were extracted.

3.2 Stage 2 – Structural Modelling and Analysis of Spillway-Gate-Reservoir Assembly

- Generate 3D FE model of the gate elements using SAP2000 and incorporate the hydrostatic and hydrodynamic loads to the gate skin
- Apply seismic loads using the time histories recorded at the pin and the spillway base (from Stage-1) and determine the demands at the gate elements
- Compute demand-over-capacity ratios of the gate members and compare the performance against the acceptance criteria developed for the gate

3.2.1 Structural Analysis Model

The structural modelling of the gate, the spillway and the reservoir is conducted using the CSI software *SAP2000 v21.0.2* and the models are developed in two progression levels which allowed comparison of the responses from 'Added-mass' approach and the improved spillway-gate-reservoir modelling approach.

3.2.1.1 Gate-only Model with Added-Mass Approach

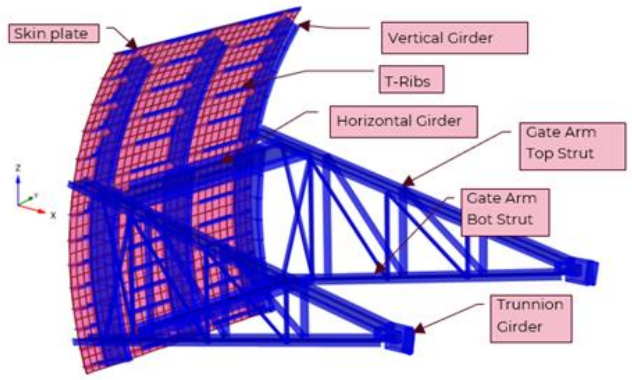


Figure 5 : Analytical model of the gate (SAP2000)

The Gate is modelled in isolation to the spillway block and the underlying soil as shown in Figure 5. Frame elements are used to model horizontal struts, vertical struts, gate arms and T-ribs. The skin plate is modelled as area element. The gate struts are connected to the trunnion pins by means of rigid elements. The trunnion pins are restrained for translation in upstream-downstream (UX), cross-channel (UY) and up-down or gravity (UZ) directions and against the rotations RX and RZ. RY rotation is allowed about the trunnion girder. Loading is applied in the stream direction (along X) being the direction of interest while the cross-channel (UY) degree of freedom for the gate displacements is constrained.

3.2.1.2 Spillway-Gate-Reservoir Model

The 'Gate-only' model is further developed by incorporating spillway block and the reservoir. The spillway block is modelled using solid elements of concrete material properties and is supported at the base with 'pinned' supports (restrained against movement in the UX, UY and UZ directions). The reservoir is simulated by means of the 'fluid-like' material model using the 3D solid elements with mechanical properties of a fluid. Considering the symmetry between the two gates, a slice of the spillway structure and reservoir equivalent to the width between the two adjacent concrete piers is modelled which includes one gate, concrete piers on each side of the gate arms, and the reservoir as shown in Figure 6.

The length of reservoir modelled is equal to four times the depth of the reservoir. Rigid links connects the reservoir surface with the gate skin plate surface in the stream direction. Reservoir elements had courser mesh below the gate sill level while finer mesh sizes were used above sill level connecting to the fine mesh of the gate skin (Wilson 1998). Reservoir side boundaries were restrained in the cross-channel (UY) direction, the base was restrained in vertical (UZ) direction while the reservoir boundary at the upstream end is allowed the top-down movement and restrained in UX and UY. The reservoir top surface is free to move vertically while the upstream boundaries are modelled as fully reflective or non-absorbing and undamped.

This modelling technique allows the model to automatically calculate the hydrostatic and hydrodynamic loads generated due to reservoir-gate interaction. Analysis is conducted along the stream direction using nonlinear direct integration time history (NLTH) load cases with constant 5% viscous and modal damping ratio. Sensitivity analysis on the output sampling rate was conducted and output time-step of 0.01 sec. was adopted to generate the time history responses.

The absolute displacement formulation of the structural dynamics is used to apply seismic loads in the stream direction. The displacement time histories at the gate trunnion and the spillway base were used as input seismic loads allowing the hydrodynamic pressures to build-up through pushing the spillway and the gate in and out of the reservoir along the stream direction (Muhamad Aslam et al 2002). The effect of vertical ground motion is to vary the pressure of the fluid during the seismic excitation. The vertical acceleration time histories were concurrently applied with their horizontal component along the stream.

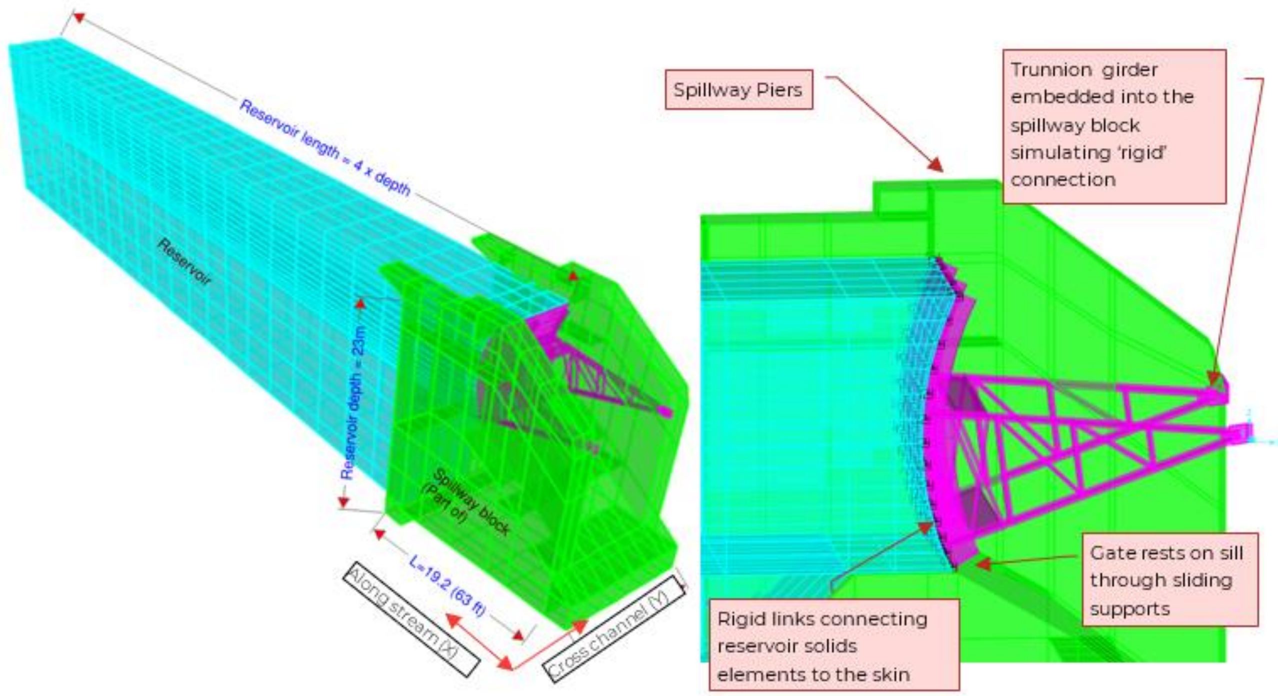


Figure 6 : Analytical model of Spillway-Gate-Reservoir [L] 3D sectional view near the gate [R] (SAP2000)

4 RESULTS AND DISCUSSION

Using the structural analysis models thus developed in Stage-2 the results between the 'added-mass' versus 'two approaches using the respective models are compared to verify the outcomes and understand the effects of the two analysis methods.

4.1 Modal Response

Modal analysis results from the 'Gate-only' and the 'Spillway-Gate-Reservoir' models are summarized in Table 1 to understand the response of each component and the integral assembly. The 'Wet' and 'Dry' modal periods refer to the periods 'with' and 'without' the water present respectively, while values in brackets are the associated percentage mass participation ratios.

The modal periods from the 'dry' gate and the spillway were reproduced in both models of 'Gate-only' and the 'Spillway-Gate' which indicate proper connection between the steel gate and the concrete spillway. The first two fundamental translational modes of the spillway-gate assembly, where they move towards the reservoir, are responsible for most (approximately 80%) of the total hydrodynamic force on the gate. The dry spillway-gate period for the first mode shape was calculated from the formulation proposed by (Fenves & Chopra 1987) to be 0.067 seconds which correlates well with the model results.

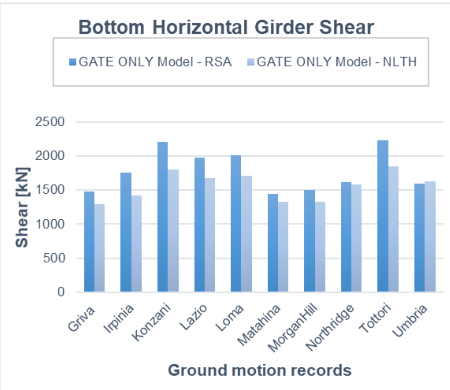
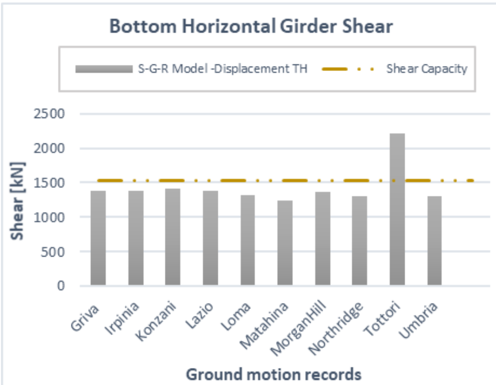
Table 1 : Summary of modal periods of the spillway block and the gate in the stream direction

S. No	Response Category	Gate-only model	Spillway-Gate model	Spillway-Gate-Reservoir model	Remarks
1	Dry gate period $T_{G\ dry\ 1}$	0.065 (16.3)			Dry gate and spillway block periods are similar
2	Dry gate period $T_{G\ dry\ 2}$	0.032 (50)			
3	Wet gate period $T_{G\ wet\ 1}$	0.24 (81.5)			Reservoir simulated as added masses
4	Dry spillway-gate period $T_{SG\ dry\ 1}$		0.0577 (42)		Similar periods replicated in the Full model. Subscripts 1 and 2 denotes first symmetric and antisymmetric modes
5	Dry spillway-gate period $T_{SG\ dry\ 2}$		0.0254 (26)		
6	Wet spillway-gate-reservoir period $T_{SGR\ wet\ 1}$			0.0579 (12)	1st symmetric mode when spillway-gate moves towards the reservoir
7	Wet spillway-gate-reservoir period $T_{SGR\ wet\ 2}$			0.0242 (10.5)	1st antisymmetric mode when spillway-gate moves towards reservoir
8	Reservoir horizontal period $T_{R\ 1}$			6.75 (47)	
	Mass distribution (%) Gate = 22.4 tons Spillway Block = 6405 tons Reservoir = 15148 tons	Gate = 100%	Gate = 0.36% Spillway = 99.6%	Gate = 0.1% Spillway = 29.7% Reservoir = 70.2%	

4.2 Gate Trunnion Reactions and Girder Demands

Peak trunnion reaction and horizontal girder shear forces demands were selected for comparison (Table 2) as the preliminary assessment of demand over capacity ratios found these elements as the weakest link.

Table 2 : Result comparison between the Gate-only and the Spillway-Gate-Reservoir models

Parameter	Gate-only 'Added mass' model	Spillway-Gate-Reservoir model
Trunnion Reaction (per trunnion pin)	Hydrostatic Load = H = 1484 kN Avg. Hydrodynamic Amplification: RSA Method = 2.4	Hydrostatic Load = H = 1564 kN Avg. Hydrodynamic Amplification: Displacement based NLTH = 1.77
Horizontal Girder Shear Force (Bottom)	 Avg. Hydrodynamic Amplification: Using Response Spectrum Method = 2.2 Using linear acceleration TH = 1.9	 Avg. Hydrodynamic Amplification: Using displacement TH at base of spillway = 1.38

5 PERFORMANCE EVALUATION

5.1 Demand over Capacity Ratio

The average demand over capacity D/C_{INEL} ratios together with the assessed post-earthquake performance for the key gate members are presented in Table 3. The D/C_{INEL} ratios are calculated as below;

$$D/C_{INEL} = \frac{\gamma_i D_{i\,n} \times {}^{S_p}/_{\mu_j}}{\emptyset C_n} \geq 1.0 \qquad (2)$$

Where;

$\gamma_i D_{i\,n}$ = imposed average demand(s) 'n' under the SEE event loading condition 'i',

$\emptyset C_n$ = nominal (elastic) capacity including appropriate resistance factors,

μ_j = ductility capacity based on the member category and the associated failure mode 'j'.

From the D/C ratios calculated above, it is established that all gate members including the anchorage have enough capacity to withstand the demands imposed during the 1 in 2500 year SEE event and are expected to remain elastic.

Table 3 : D/C ratio for 1 in 2500 AEP seismic event

Member	**Mode**	**Avg. D/C_{INEL} ratio**	**Response Type**	**Assessed effect**
Horizontal Girder	flexure	0.59	Flexure yielding	Negligible. Resulting deformations unlikely to cause significant leakage or prevent the gate being lifted.
	shear	0.93	Shear deformation (and possibly fracture) of the girder web between arm and vertical girder.	Potentially significant. Associated gate distortions could affect gate operation.
Vertical Girders	combined (flexure + Compression)	0.52	Flexure yielding	Negligible
	shear	0.60	Shear deformation of web between skin-plate and horizontal girder	Potentially significant. Distortions in the gate skin geometry could affect gate seals and possibly impact the gate operability
Gate Arm	axial compression	0.44	Compression buckling of gate arms	Significant impact on overall gate performance including structural instability, significant leakage and gate inoperability.
	combined (compression + bending)	0.71	Compression buckling of gate arms & local flexural yielding	(as above)
Girder to Arm Bolted Connection	bolt tension elongation	0.5	Bolt elongation and rupture	Not critical. Large plastic deformations in the horizontal girder are needed to strain the bolt beyond its rupture limit

5.2 Failure Hierarchy

To understand the progression of damage and the damage mechanisms that the key gate members are expected to have, the failure hierarchy is studied in case the imposed seismic loads exceed the nominal yield capacities of members. The aim of failure hierarchy is not to predict the sequence of events but to provide useful information for considering strengthening options to improve the resilience of the gates. The hierarchy of failure for the gate structure is presented in Figure 7.

Figure 7 : Critical components in the hierarchy of gate failure

From the schematic in Figure 9, the critical component governing the overall gate response is the brittle failure of the horizontal girder under the shear demands near end supports.

The capacity of gate anchorage assembly is governed by the trunnion girder shear strength under the reactions from compression loads in the arms. The trunnion strength is less than the gate arm total buckling failure capacity when conservatively assuming simultaneous buckling of top and bottom gate arm struts. However, it is sufficient to sustain the imposed demands under 1 in 2500 AEP seismic event. It is likely that in an event larger than 1 in 2500 AEP the gate members will reach their respective yield strength and have the ability to deform in a ductile manner and thereby prevent loading of the connection region beyond its nominal capacity.

Next in the failure hierarchy is the bottom strut of gate arm when loaded with end-moments from horizontal girder's bolted end connections together with the axial compression. The governing failure mechanism of the arms is likely to be the axial compression buckling of the struts which are expected to buckle in a ductile manner due to the low slenderness ratio through local yielding under the combined actions.

The bolted end-connection between the horizontal girder and the arm struts provides end-fixity to the girders and imparts end-moments into the gate arm struts through tension elongation in the outermost 4 nos. fitted bolts. Our assessment indicates that even when the horizontal girder end-rotation reaches its peak rotation capacity of $\theta_y + \theta_p = 0.030$ radians (corresponding mid-span deflection of 100mm), the imposed elongation strain on the bolts is approximately 14%, which is less than the bolt rupture strain capacity of 20% as per BS 1083:1951. Since the assessed horizontal girder response under SEE event remains within the elastic range, the bolts are unlikely to reach or exceed their yield elongation limits.

6 CONCLUSIONS

The comparison of gate member responses for the trunnion reaction and the horizontal girder shear show significantly large demands when using the gate-only added mass approach compared to the spillway-gate-reservoir model. The differences are attributed to two reasons;

- possible conservatism in the added mass approach because the interaction of the fluid with the gate is discounted by forcing the hydrodynamic component to act in-phase with the same frequency as the gate. In reality, the reservoir and the spillway-gate assembly interact based on their dynamic properties, which in this case is achieved by using 'fluid-like material' approach in the Spillway-Gate-Reservoir model.
- the response spectrum analysis generally overestimates the seismic demand compared to time history analysis due to its fundamental procedure of using peak accelerations and combining vibration modes and orthogonal components in a conservative way using CQC and SRSS methods.

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