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Response-History Analysis of the Moondarra Intake Tower: A Novel Approach Based on New Methodologies and Performance Requirements

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ABSTRACT:
A nonlinear response-history analysis was undertaken to assess the stability and strength of Moondarra Intake Tower. The objectives of the study included determining: a) whether the tower could withstand the design earthquake without collapse; b) whether remedial works would be required; and c) whether the remedial works could be performed without affecting the operation of the tower. A response spectrum analysis conducted previously indicated that the tower would not withstand the seismic demand in shear and bending. The first difference of the present approach relative to conventional response analyses is the use of the state-of-the-art Conditional Mean Spectrum (CMS) as the target spectrum for development of ground motions, as opposed to the conventional Uniform Hazard Spectrum (UHS). The second major difference is the estimation of crack size to determine potential for leakage to the interior of the tower and whether is safe to undertake repair works from inside the structure without a reservoir drawdown. The results showed that the tower can safely withstand the design earthquake, with no significant structural damage. The size of the cracks would be small and the repair works can be safely undertaken from inside the tower while remaining operational. No upgrading measures are therefore required.

Keywords: Intake-tower, seismic, response-analysis, nonlinear, CMS

1. INTRODUCTION

The Moondarra Reservoir belongs to Gippsland Water (GW) and is located 160 km east of Melbourne, in Victoria, on the SE corner of Australia. The intake tower, built in 1962, is a 40.8 m high, dry well tower, with an internal diameter of 4.5 m and four draw-off ports which enable water to be drawn from different levels within the reservoir. A previous response spectrum analysis of the tower based on the procedures recommended by the USACE (2003 and 2007) indicated that the tower was not able to withstand the Maximum Design Earthquake (MDE) with 1 in 1,000 Annual Exceedance Probability (AEP), for which bending and shear failure of the tower shaft was expected. The schematic definition of the components of the tower is presented in Fig. 1.

The objective of the nonlinear response-history analysis was to determine: a) whether the tower could withstand the design earthquake without collapse; b) whether remedial works would be required; and c) whether the remedial works could be performed without affecting the operation of the tower. For this study the intake tower was subjected to 1 in 1,000 AEP ground motions acting concurrently with the reservoir at Full Supply Level (FSL), or RL 166.16 m.

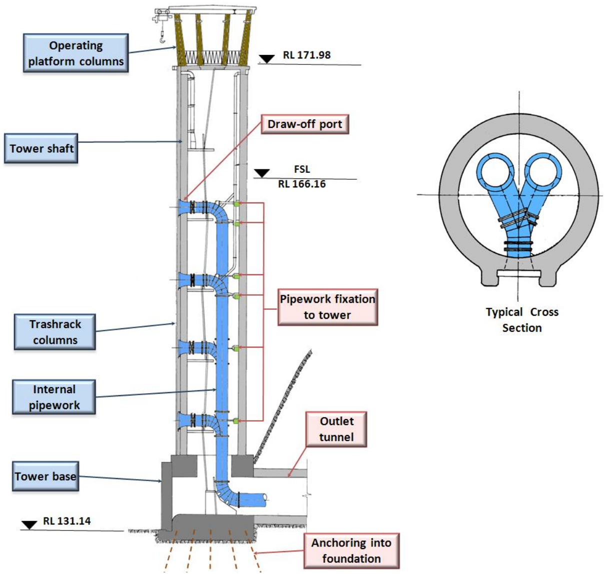

Figure 1. General arrangement of the tower

2. SEISMOLOGY AND SEISMIC ASPECTS

In a regional basis, most earthquake activity in Australia is considered “intra-plate” seismicity.

Current local stresses in the area of the Moondarra Reservoir are thought to be dominated by NNW-SSE regional compression which commenced in the Late Tertiary (after deposition of the brown coal reserves in the nearby Latrobe Valley). The nearby monoclinal folds are the surface expression of reverse faults in the basement rocks of the area. These faults may represent the relief stress axes in the global tectonic process characteristic for this part of Victoria.

2.1. Conditional Mean Spectrum (CMS)

Accelerograms for the two orthogonal horizontal directions were developed using the spectral matching technique. Instead of the conventional UHS, the state-of-art Conditional Mean Spectrum (CMS) (Baker, 2011) was generated and used as target spectrum.

According to Baker, accelerograms for response-history analyses are often obtained by selecting ground motions that match a target response spectrum. The commonly used UHS has been shown to be an unsuitable target for this purpose since it conservatively implies that large-amplitude spectral values will occur at all periods within a single ground motion. The alternative proposed by Baker provides the expected (mean) response spectrum, conditioned on occurrence of a target spectral acceleration value at the period of interest. It is argued that this is the appropriate target response spectrum for the goal described above, and is thus a useful tool for selecting ground motions as input to the dynamic analysis.

For this study, a CMS for 1 in 1,000 AEP was generated and used as the target spectrum for the development of accelerograms for the nonlinear runs. In accordance with the methodology, the CMS was produced around a single structural period that matched the fundamental period of the intake tower (0.55 s, when the reservoir is at FSL).

Fig. 2 presents the comparison between the conventional UHS and the CMS. It can be observed how both spectra coincide at the specified period of 0.55 s, but for all other periods the CMS ordinates are less than these of the UHS.

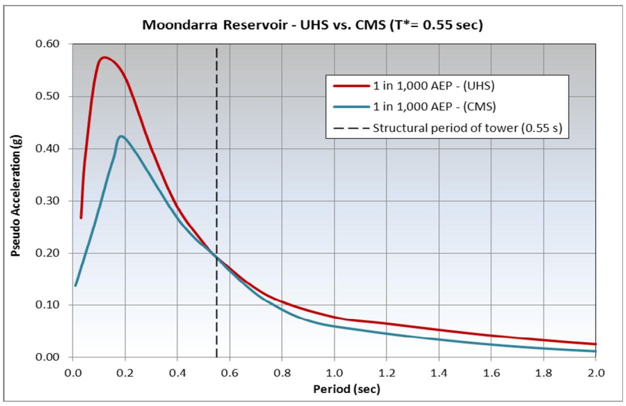

Figure 2. Comparison of response spectra: UHS vs. CMS

2.2. Generated Ground Motions

The spectral matching technique was used to generate acceleration-time series. In the absence of specific strong motion data for the Moondarra Reservoir region, it was necessary to adjust 'foreign' accelerograms imported from worldwide strong motion databases. Table 1 presents selected ground motions for this study, including their magnitude (Mw) and distance to the epicentre.

Table 1. Selected accelerograms for spectral matching

Earthquake	Station Name	Year	Mw	Distance (km)
Northridge	PAR	1994	6.7	19.3
Cape Mendocino	PET	1992	7.0	11.9
Friuli SRO	SRO	1976	5.5	20.1

Fig. 3 presents the 1 in 1,000 AEP accelerograms for the two horizontal components of the PAR ground motion (Northridge, 1994) generated for the nonlinear runs of this study.

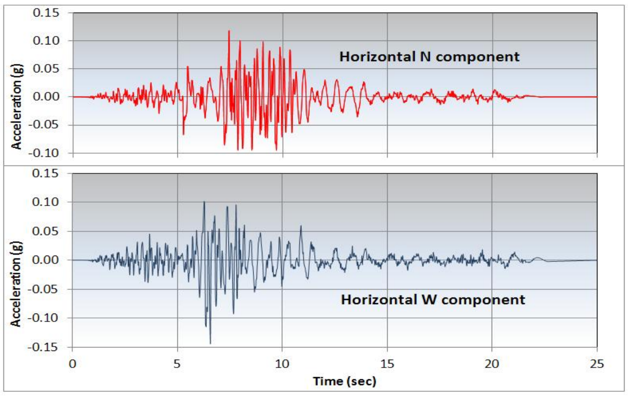

Figure 3. Spectrally-Matched Accelerograms for PAR ground motion (Northridge earthquake, 1994)

3. FINITE ELEMENT ANALYSIS

The Finite Element (FE) modelling of the intake tower was carried out using SeismoStruct v5.0.5 software (SeismoSoft, 2010). The program uses a fibre approach to represent the cross-section behaviour, for which each fibre is associated with a uniaxial stress-strain relationship. The sectional stress-strain state of beam-column elements is obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibres.

3.1. Finite Element Model

The finite element of Moondarra intake tower was composed of 120 inelastic frame elements representing the tower base, the tower shaft, the columns of the operating platform and the trash rack columns. The internal pipe work was represented by 53 elastic frame elements.

The frame elements explicitly model the shaft section, which is 0.68 m thick, with 24 mm dia. longitudinal bars (48 in the interior face and 69 in the exterior face) and 19 mm dia. horizontal hoops spaced at 0.38 m.

The internal pipework is attached to the internal face of the shaft at the same levels of the draw-off ports and fixed at the bottom slab of the tower. There is a significant degree of restraint to both displacement and rotation of the tower at the base level resulting from the embedment of the structure into sound rock, the anchoring of the base of the tower into the foundation rock and the structural connection of the tower with the outlet tunnel. Consequently, the model of the tower has been assumed fully fixed at the base. Fig. 4 presents the rendered view of the frame elements of the model developed for this study.

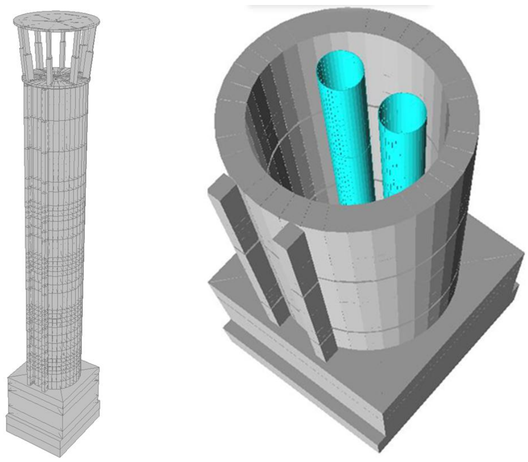

Figure 4. Full and truncated views of the tower's FE model

3.1.1. Material properties

The material properties used for the FE modelling of the intake tower were mostly taken from the 1959 "Specifications of the Project" and the 1962 "Technical Record of the Design and Construction of Moondarra Reservoir". Table 2 presents a summary of the properties used in this study.

Table 2. Dynamic Material properties for FE analysis

Material	Property	Value
Unconfined concrete	Compressive strength, f'_c	25 MPa
	Initial modulus of elasticity, E_1	25,280 MPa
	Post-peak modulus of elasticity, E_2	10,000 MPa
	Residual strength, f_{c2}	5 MPa
Confined concrete	Compressive strength, f'_c	25 MPa
	Tensile strength, f_t	2 MPa
	Strain at peak stress, ε_c	0.002 mm/mm
	Confinement factor, k_c	1.0
Reinforcing steel	Modulus of elasticity, E_s	200 GPa
	Yield strength, f_y	230 MPa
	Strain hardening	0.5%
Concrete/rock interface	Angle of friction	40°
	Cohesion	Nil

3.1.2. Structural damping

Structural damping of 5% has been often adopted for the seismic analysis of hydraulic structures in general and intake towers in particular, for practicality and due to the lack of more specific data. However, tests conducted by A. Chopra in the freestanding intake tower of San Bernardino (Rea et al, 1975) indicated that damping for this type of structures is, at most, 2% for the fundamental structural period. Accordingly, a conservative damping of 1% was adopted for this study.

3.1.3. Hydrodynamic effects

The dynamic interaction between the reservoir and the intake tower was estimated using the added hydrodynamic masses methodology developed by Goyal and Chopra (1989). The estimated masses were attached to the model at corresponding nodes along the height of the structure.

4. PERFORMANCE CRITERIA

4.1. Structural Strength

Within the context of a fibre-based approach, material strains were used to determine the performance of the reinforced concrete. Different conditions of structural damage can be identified when the strain damage indicators presented in Table 3 are exceeded during the analysis.

Table 3. Damage indicators and strain limits

Damage Indicator	Strain limit
Cracking of cover concrete, ε_c	+0.0001
Yielding of steel, ε_{sy}	+0.00115
Fracture of steel, ε_{su}	+0.02
Spalling of cover concrete, ε_{cu}	-0.002
Crushing of core concrete, ε_{cu}	-0.004

Once the local condition of the structural elements was determined, the global strength was investigated by assessing the occurrence, location and extent of plastic hinges. Formation of plastic hinges was monitored by tracking the transient response of the reinforcing bars, specifically searching for generalised yielding of the sectional reinforcement at critical locations of the tower.

Finally, the analysis was validated by checking that shear failure has not occurred. For this assessment, the shear demand computed by SeismoStruct was compared with the expected shear capacity calculated in accordance with USACE (2003 and 2007), for both the diagonal shear tension and shear along a fully cracked horizontal plane.

4.2. Serviceability and Crack Width

Due to operational constrains, GW expressed the following serviceability requirements: a) any repairing works would need to be carried out from inside the structure only; and b) any repairing works proposed must be feasible, effective and safe to implement. These serviceability constraints imply that the structural damage has to be limited (no structural collapse), and that the width of the cracks has to be contained so that commercial repairing products and techniques could be used. GW also indicated that the operating platform was not indispensable for the post-earthquake operation of the tower and that the structure could be accessed after the earthquake via the outlet tunnel and /or by boat.

Since concept of a “reparable crack size” is rather abstract and debatable, a first approach consisted of assessing the crack size against a limiting value that is considered appropriate for serviceability. The Australian Standard for Concrete Structures for Retaining Liquids (2001) establishes that durability performance in concrete is achieved if crack sizes in continuously submerged structures are less than 0.10 mm (in tension) or 0.15 mm (in flexure). The American Concrete Institute (2007) establishes that cracks up to 0.13 mm wide may heel in the presence of moisture. Consequently, a crack size of 0.13 mm was adopted as the initial performance threshold for serviceability. If the estimated crack size was less than this limiting value, no repair works would be required.

The transient tensile strains in the external cover concrete of the tower were used as an indicator to determine residual the crack sizes.

4.3. Structural Stability – Rocking and Sliding

The global stability of the intake tower, in terms of sliding and rocking, was assessed in a separate model using the program RS-DAM (Leclerc et all, 2002). Both conditions are considered quite unlikely to occur due to the actual degree of restraint of the tower base. However, a conservative assessment was performed assuming the tower as a freestanding rigid structure detached from the base, whose stability relied solely on the friction between the concrete of the foundation slab and the rock.

5. NONLINEAR RESPONSE-HISTORY ANALYSIS

The inelastic response-history analysis of Moondarra Intake Tower was carried out for the three developed ground motions: PAR (Northridge earthquake, 1994), PET (Cape Mendocino earthquake, 1992) and SRO (Friuli earthquake, 1976). All ground motions produced in similar structural response. Due to space limitations only selected results from the PAR ground motion (Northridge earthquake, 1994) are presented graphically in this paper.

5.1. Structural Strength

Transient stress and strain of materials at selected points of the tower are presented on Fig. 5, for the PAR ground motion. The top graph displays the maximum tensile stress on any of the reinforcing bars of a selected cross section of the tower, while the bottom graph display the maximum compressive strain on cover concrete. The location of the selected points for presentation of results is also presented on the right bottom corner of Fig. 5.

5.1.1. Yielding of reinforcement steel

For ease of understanding, the yielding of reinforcement steel was assessed in terms of stresses rather than strains, that is, by comparing the resulting stress in the bars vs. the yield strength of the reinforcement (230 MPa). The resulting stresses in the reinforcement were calculated using Hook’s law, by multiplying the resulting strains by the modulus of elasticity of the steel.

For all three ground motions, the maximum tensile stresses in the reinforcing bars were less than the yield strength of 230 MPa. The maximum tensile stress on the reinforcing steel at the base of the shaft section (B7) was 95 MPa, for the SRO ground motion, while the maximum demand at the top of the tower shaft (B38) was 90 MPa for the PAR ground motion.

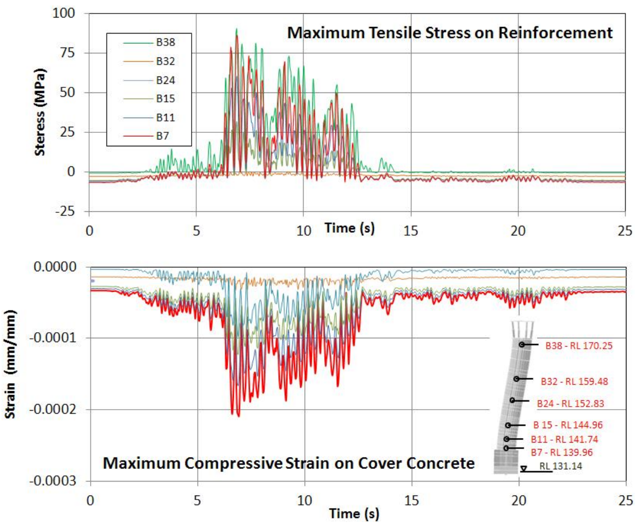

Figure 5. Tensile stress and compressive strain at selected locations of the tower, PAR ground motion - Northridge (1994)

Based on the observed distribution and magnitude of the tensile stresses in the reinforcement steel, no yielding, fracture of steel or formation of plastic hinges was anticipated in the tower shaft.

5.1.2. Spalling and crushing of concrete

The compressive strains in the cover concrete were in all cases less than the spalling indicator strain of -0.002 mm/mm. The maximum recorded compressive strain in the cover concrete was -0.00022 mm/mm for the SRO ground motion, which occurred at the base of the shaft section (B7). The compressive strains were also in all cases less than the indicator strain of -0.004 mm/mm for crushing of the concrete core. The maximum recorded compressive strain in the concrete core was -0.0002 mm/mm for the PAR ground motion.

Based on the observed distribution and magnitude of the compressive strains in the concrete, no spalling of cover concrete or crushing of confined concrete was expected.

5.1.3. Shear force

Fig. 6 presents the resultant of absolute shear demand for the PAR ground motion, at selected level RL 138.96 m (B7). The structural demand is compared with the estimated shear capacity of the tower for both diagonal shear tension and shear strength along a horizontal plane. The shear demand in the tower shaft was in all cases less than the shear capacity in diagonal tension and the shear capacity along a horizontal crack.

The maximum transient shear demand recorded was 2,803 kN for the PAR ground motion, occurring at the base of the shaft section (B7). This demand is less than the estimated shear capacity in diagonal tension of 9,200 kN and also less than the transient shear capacity along a horizontal crack of 7,020 kN.

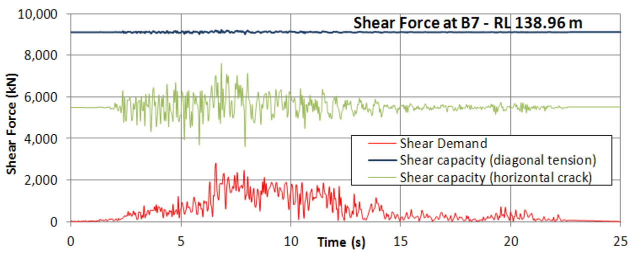

Figure 6. Shear capacities vs. shear demand, PAR ground motion - Northridge (1994)

Based on the observed ratio between the demand and the structural capacity, no shear failure of the shaft section of the tower was anticipated.

5.2. Serviceability - Crack Size

Fig. 7 presents the plot of crack sizes on any location of a selected cross section of the tower during the PAR ground motion, and the residual post-earthquake crack size, at selected levels of the structure. The crack sizes were calculated using the resulting tensile strains of the cover concrete and compared with the damage control crack size of 0.13 mm.

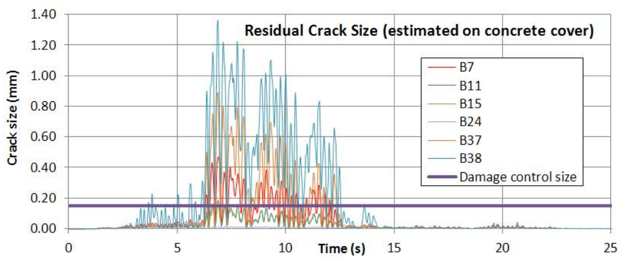

Figure 7. Crack size during and after the ground motion, PAR ground motion – Northridge (1994)

For all three ground motions, cracking during the ground shaking is expected to occur along the height of the tower shaft section.

At the base of the shaft (B7), where cracking would be more critical due to the depth of water, the maximum estimated crack size during the ground motion was 0.51 mm, for the SRO ground motion. At the same level, the maximum crack size during the PAR ground motions was 0.45 mm. At the top of the shaft (B38), a less critical location, the maximum crack size during the ground motion was 1.54 mm, for the SRO ground motion. At the same level, the maximum crack size during the PAR ground motion was 1.35 mm.

In all cases once the structure came to a rest after the shaking, the cracks closed due to gravity loading and the residual size of the cracks reduced to less than 0.001 mm. This residual crack size is smaller than the minimum serviceability requirement of 0.13 mm.

Based on the expected residual crack size, minimal leakage through the cracks is expected. In accordance with previously observed behaviour of concrete structures (ACI, 2007) it is likely that cracks with such small residual size heal by themselves with time. If required, remedial works on cracks this size could be completed safely from inside the tower; however such work would only be necessary for cosmetic reasons.

5.3. Stability Assessment

5.3.1. Rocking analysis

The transient the position of the resultant in respect to the tower's base edges is presented in Fig. 8 for the PAR ground motion. The 50% mark represents the centreline of the tower's base, while the 0% and 100% marks denote the edges of the base.

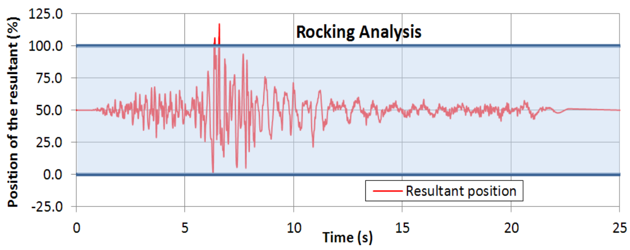

Figure 8. Rocking analysis, PAR ground motion – Northridge (1994)

For the duration of all three ground motions the position of the resultant was mostly located within the base, with only limited excursions outside the base. These excursions included maxima of 25% out of the base for the PET ground motion and 17 % for the PAR ground motion. The estimated transient factor of safety against overturning was also momentarily reduced to less than 1.0. These peak overturning actions occurred for only a fraction of a second in each cycle, with intermediate opportunities to unload. This is the reason why in all cases there is no record of actual rotation of the rigid block.

Based on the results of the rocking analysis the tower is expected to be stable against overturning. The tower does not need to rely on the anchorage to the foundation rock or on the structural connectivity to the outlet tunnel for rotational stability. Therefore these connections are not expected to suffer significant damage due to rocking of the tower.

5.3.2. Sliding analysis

The estimated seismic demand on the tower under all three ground motion was not large enough to produce sliding at the foundation level. The lowest transient factor of safety against sliding was estimated for the PET ground motion (FS= 2.4), and consequently no transient or residual displacements were predicted.

Based on the results of the sliding analysis the tower is expected to be stable relying solely on the frictional resistance between the concrete base and the foundation rock, without overstressing the anchorage to the rock or the structural connectivity to the outlet tunnel. These connections are not expected to suffer significant damage due to sliding of the tower.

6. CONCLUSIONS

Three ground motions with 1 in 1,000 AEP were used to model the nonlinear response-history of Moondarra Intake Tower. The assessment employed the state-of-the-art CMS for development of the ground motions and accounted for project-specific serviceability constraints which set the performance requirements for the structure.

The developed CMS matched the conventional UHS at the fundamental structural period, but was less conservative for all other periods, most notably in the range of 0.01 to 0.4 seconds. In general, the adoption of the CMS method for the development of the accelerograms for this study resulted in an overall smaller demand on the tower, when compared with the one that would have resulted if the conventional UHS method were used.

The serviceability constraints of the project and practical remedial work considerations implied that the acceptable performance of the tower, in terms of structural damage, would consist of residual crack sizes limited to 0.13 mm.

This study found that the tower is expected to safely withstand all three ground motions. No structural collapse is anticipated, since damage indicators showed that yielding of reinforcement, formation of plastic hinges and spalling or crushing of the concrete are highly unlikely. The tower was also found to be safe against both sliding and overturning.

The damage to the shaft of the tower is expected to be limited to cosmetic damage, with residual cracks of less than 0.01 mm wide forming along the full height but concentrating mainly at the base and top of the shaft. Due to their small widths the cracks are not expected to leak, and if they do, they are likely to be self-healing. Remedial works on cracks this size could be completed safely from inside the tower.

Since the structure was found able to withstand sliding and rocking actions as a freestanding tower, the structure does not rely on its connections to the foundation (via anchoring) and to the outlet tunnel (via reinforcing bars) to be stable. Thus, these connections are not likely to suffer significant damage during the ground motions.

This paper presented the benefits of performing a sophisticated response-history analysis in combination with a novel approach such as the CMS methodology and project-specific serviceability requirements. The results of this study showed that the seismic behaviour of the structure is indeed satisfactory and that no upgrading works are necessary for a tower that was previously deemed as structurally inadequate when assessed using a response spectrum analysis.

For Gippsland Water, the results of the present study represent important potential savings in capital works and business disruption that, otherwise, would have been needed if upgrading works were required at this stage.

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