Development of seismic design parameters for the Maari Wellhead Platform deck

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ABSTRACT: This case study concerns the development of seismic design response parameters at the deck level of the Maari Wellhead Platform (WHP) which is located 80 km off the South Taranaki coast in New Zealand. These parameters have been derived as a basis for future seismic design of secondary or appurtenance structures during the operating life of the WHP. The purpose of this paper is to provide a detailed summary of the challenges encountered and adopted methodology in the: (i) development and validation of the WHP analytical model, including conclusions drawn to explain the differences between predicted and observed fundamental vibration periods; (ii) appropriate selection and scaling of earthquake-induced ground motion records to perform linear elastic response-history analysis of the structure; and (iii) derivation of pseudo-acceleration response spectra at the WHP deck level. The dynamics of the WHP structure are also discussed as they are integral to rationalising the observed deck response from the response-history analyses.

1 INTRODUCTION

1.1 Description of the Maari Wellhead Platform

The Maari Wellhead Platform (WHP) is located in an oilfield 80 km south off the South Taranaki coast in New Zealand. The facility has been operational since 2009 under OMV New Zealand Ltd. (OMV) and was designed by Arup Energy Clough (Arup). Figure 1 illustrates the Maari WHP platform during: (a) transportation to the oilfield by wet-towing; and (b) in the final stages of installation. The following important features can be observed:

- The deck (50 m x 55 m with a 29 m x 25 m wide opening to accommodate the jacket) is a barge-type structure that provides support to the topsides equipment (e.g., crane pedestal, heli-deck, accommodation block, drilling rig etc.) and buoyancy for transportation and installation.
- The jacket (22 x 26 m with a height of 131 m) consists of four tubular chords connected in a lattice structure. Between the seabed and underside of the deck, the lattice is split into five bays with each bay having cross-braces traversing the four vertical sides. Horizontal cross braces are located between each bay.
- The base (44 m x 48 m with 6 m deep skirts), although not visible in Figure 1, is a rectangular skirted steel gravity structure for founding the structure and its installation is assisted by suction.

The purpose of the structure is to provide an elevated platform from which the drilling and maintenance activities take place. Fixed to the platform are the topsides equipment (appurtenances) which are essential to the platform operation. This includes structures such as the drilling derrick, crane, lifeboats, helideck, and staff quarters.

1.2 Seismic design aspects

In the seismic context, the main platform structure must be designed for earthquakes in a similar way to on-shore structures, and the appurtenances must be designed similar to 'parts and components'. The deck motions can be much higher than those experienced at the sea floor, which is not unexpected given that the response of the main structure changes the amplitude, frequency content, and duration of earthquake shaking at the seabed. In this way, the main structure acts as a filter between the appurtenances and the seabed.

Many of the appurtenances are very stiff which means they can be treated as rigid bodies that experience the same accelerations as the deck; however, a number are sufficiently flexible that they have their own dynamic response to the deck motion. While Section 8 ('Requirements for Parts and Portions') of NZS 1170.5:2004 (Standards New Zealand, 2004) considers flexibility of parts in the spectral shape coefficient, this coefficient has been developed for response of a typical onshore building and its application to an offshore oil platform may not be reliable. In this case, it is necessary to develop deck response spectra to properly capture the effect of excitation of the appurtenance to the expected deck motions.

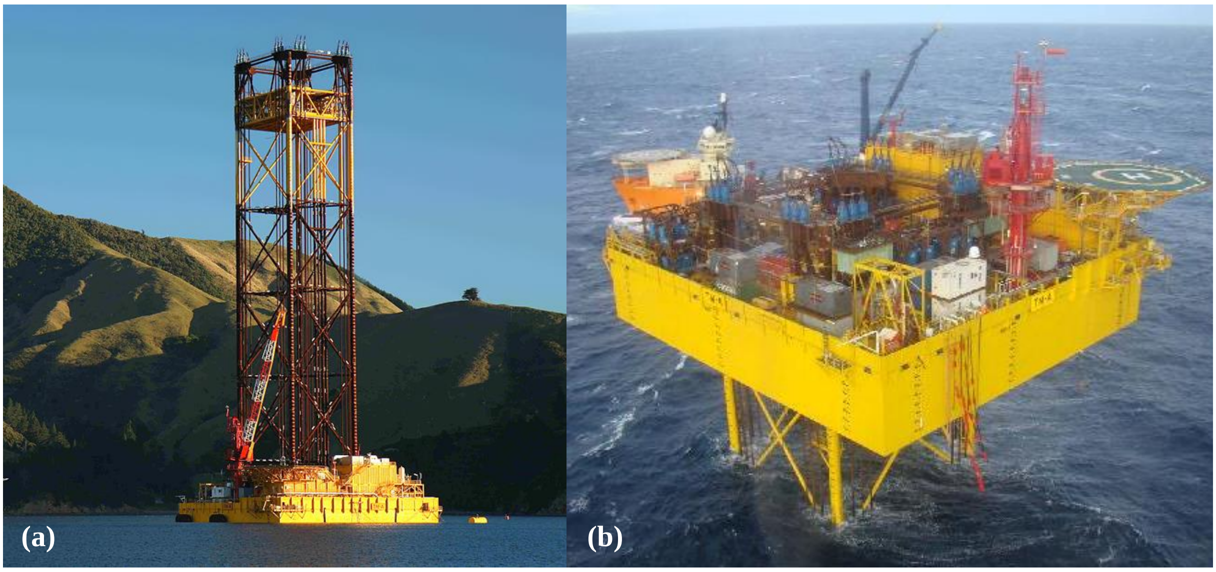

Figure 1: Photographs of the Maari Wellhead Platform taken during: (a) transportation to site (after Arup (2015)); and (b) operation after installation on site (after Caprari Pumps (UK) Limited (2009)).

1.3 **Response to 2013 Cook Strait earthquakes**

The 21 July 2013 M_w6.5 Seddon earthquake (located approximately 200 km south-east from the WHP site) provided an opportunity to observe the response of the platform to a significant earthquake. OMV had previously installed on the deck two accelerometers, from which a good record of deck accelerations was captured. There was also anecdotal evidence that the response of some appurtenances was significant, despite the modest shaking that might be expected from a relatively remote earthquake.

OMV initially engaged Bimaris Limited (Bimaris) to process the motion data to determine the maximum platform displacements during the earthquake. From this information, the primary natural periods of the platform were extracted. The derived natural periods were surprising in that they were about 30% less than those estimated at the design stage. While such differences might be expected, OMV engaged Beca to re-check the seismic design basis for the platform and develop new deck response spectra given the new information provided by the Seddon earthquake.

1.4 **Scope of assessment**

OMV asked Beca to investigate a number of seismic aspects for the platform:

- Develop a detailed analysis model of the platform to investigate the natural frequencies of the structure in an effort to understand the difference between predicted and observed natural periods. This model was based on more accurate operating mass than would have been assumed at the design stage.
- If required, calibrate the model to the observations (i.e., stiffen the structure to match the observed periods), and develop new deck response spectra based on the calibrated model.

This paper provides a summary of the adopted methodology and challenges encountered in the: (i) development and validation of the analytical model, including conclusions drawn to explain the predicted vs. observed period differences; (ii) appropriate selection and scaling of earthquake-induced

ground motion records to perform linear elastic response-history analysis of the structure; and (iii) derivation of pseudo-acceleration response spectra at the WHP deck level. The dynamics of the WHP structure are also discussed as they are integral to rationalising the observed deck response from the response-history analyses.

2 ANALYTICAL MODELLING

2.1 Model summary

The derivation of deck response spectra for the Maari WHP requires an analytical model of the structure to be developed. The input files of a WHP model previously developed by Technip Malaysia (Technip) were provided to Beca by OMV. These files were created using the SACS offshore structural analysis and design software during a previous commission. The model included the following features:

- Foundation and soil stiffness is modelled as simplified equivalent springs applied to the base of the jacket chords.
- The jacket structure is modelled using beam finite elements.
- The deck structure is modelled using shells stiffened with beam finite elements.
- Major appurtenances are modelled as rigid tripods with lumped masses applied at the appropriate locations.

The SAP2000 structural analysis package was used in the present study due to its ability to perform dynamic response-history analysis using earthquake-induced ground motion records. In addition, the program includes an in-built response spectrum generator. Although SACS model input files are compatible with SAP2000, significant modifications and losses of information were observed following the model import process. These include, but are not limited to:

- Incomplete geometry and loss of connectivity between elements.
- Incompatible meshing between deck shell elements.
- Loss of springs at the base of the jacket structure.
- Loss of both structural and non-structural mass information

The aspects above were identified using a thorough debugging process and subsequently incorporated into the SAP2000 model.

2.2 Key modelling assumptions

It is important to note that the modelling of the WHP structure was carried out using certain assumptions which are considered to be reasonable:

- The mass of the structure includes allowance for non-structural mass, flooded members, marine growth, conductor pipes and I-tubes.
- Structural mass has been calibrated against measured mass provided by OMV.
- Marine growth allowance (mass and thickness) has been based on Arup-calculated design values as accurate records of current marine growth were not available for this assessment.
- The Morison allowance for fluid-structure interaction effects have been considered based on the effective volume of water displaced by the submerged members including the marine growth profile.
- Conductor pipes are self-supporting vertically, but are supported horizontally by the WHP. Any ‘rattling’ between the conductors and conductor guides has not been modelled.
- Soil-structure interaction has been modelled using simple linear springs originally derived at the design stage at the base of the jacket chords.
- The drilling derrick can move between 12 positions but its location in the model is set to a neutral position at the centre of the conductor pipe group.

3 COMPARISON OF NATURAL VIBRATION PERIODS

As described in Section 1, one of the main tasks of this study was to carry out a comparison of the fundamental periods of vibration in the two translational directions between the: (i) Beca SAP2000 model; (ii) Technip SACS model; (iii) Arup lumped mass model for jacket strength analysis; and (iv) periods derived by Bimaris from ambient motion and response of the platform to the 2013 Cook Strait earthquakes captured using accelerometers.

Table 1 provides a comparison of the vibration periods. Since the Beca and Arup/Technip models had the same foundation stiffness, the small difference between the periods predicted by the models could be attributed to the following reasons:

- The Arup/Technip model was based on the 'not to exceed' (upper-bound) topsides mass which is higher than the current operating weight.
- There may be differences in the allowance for hydrodynamic effects however, this aspect was not clearly documented by Arup/Technip.
- There may be differences in modelling assumptions which lead to a more flexible structure in the Arup/Technip models.

It is worth noting that for offshore platform design, longer periods tend to be critical for wave actions, which are often more severe than earthquake load cases. For this reason, there may be a tendency for designers to deliberately bias assumptions towards a more flexible structural response.

Table 1: Comparison of the first and second mode periods predicted by the Beca SAP2000 model with values obtained from Arup/Technip models and observed response of the structure.

Mode of vibration	**Period of vibration, *T* (s)**		
	Beca (SAP2000)	**Arup (lumped mass model) & Technip (SACS model)**	**Observed (Bimaris)**
N-S Translation	4.4 s	4.9 s	3.6 s
E-W Translation	3.8 s	4.2 s	3.1 s

The larger periods predicted by the analytical models in comparison with those observed during recent earthquakes are indicative of inconsistencies between design assumptions and reality. In this case, the differences can be primarily attributed to the foundation spring stiffness values originally derived during the design stage. Although details of the foundation were not provided for this study, it appears that the local stiffness of the foundation has been underestimated (i.e., it is too flexible) for transient-type loading.

The discrepancy between the predicted and observed natural periods could also be attributed to differences between actual and predicted marine growth profile. At this stage in the life of the platform, OMV advise that there is significant difference in the predicted marine growth profile compared to that observed. The effect of this is twofold: (i) the mass of the marine growth is overestimated; and (ii) the effective volume of members is overestimated, resulting in overestimated mass allowance for hydrodynamic effects. In comparison to the foundation stiffness differences, the effect of the marine growth profile difference may be relatively minor.

4 MODEL VALIDATION

A second model was created from the WHP as-built drawings using the Strand7 general purpose finite element software to thoroughly validate the SAP2000 model. Hand calculations were also performed to aid the validation process. Figure 2 illustrates the analytical models developed using both SAP2000 and Strand 7. The model validations process was carried out by means of: (i) total mass; and (ii) modal period comparisons. A brief discussion of the findings from this exercise is provided in this section.

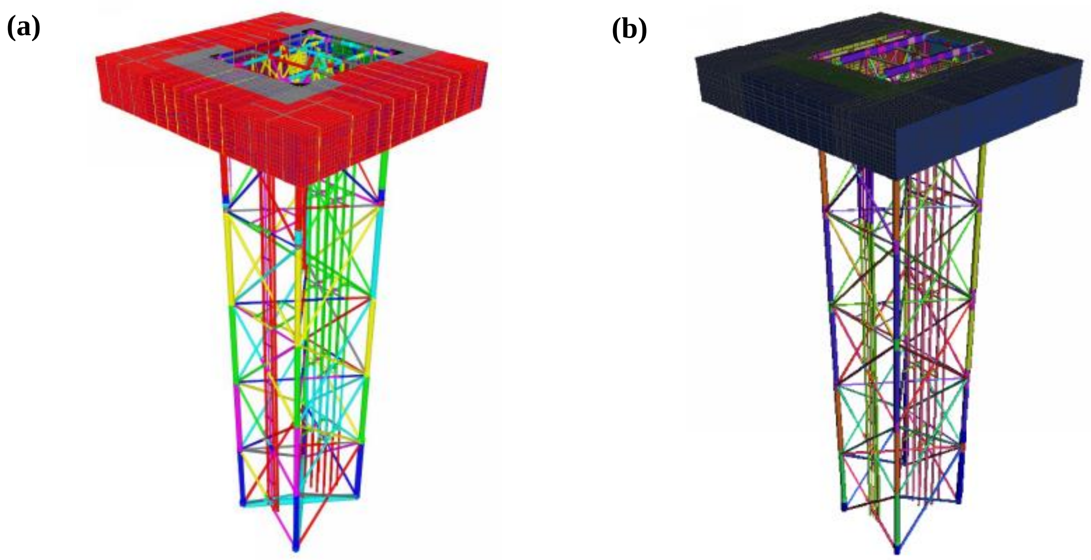

Figure 2: Analytical models of the Maari WHP developed using: (a) SAP2000; and (b) Strand7 structural analysis packages.

4.1 Mass comparison

The mass included in the models is a combination of:

- Mass data provided by OMV which contributes to the overall vertical weight of the structure. This includes the mass of the deck, jacket, appurtenances, installation equipment, and applied live loads. The mass of the base is not included.
- Additional mass due to flooded members, marine growth, hydrodynamic effects, and conductor masses.

A comparison of the above masses between the SAP2000 and Strand7 models, as well as hand calculations resulted in differences of less than 1% and were attributed to simplifications made in the marine growth profile and hydrodynamic mass allowance. This implied that the masses had been applied consistently across the primary SAP2000 model.

4.2 Modal period comparison

As discussed in Section 3, a modal period comparison between two different analytical models is a robust method of checking the dynamic properties of a structure (i.e., a good correlation between modal periods implies that the ratio of mass and stiffness properties of the models are consistent). Table 2 provides a comparison of the first two modal periods between the two models and hand calculations using base springs derived at the design stage. It can be observed that the periods are consistent with each other with a range of variation that can be expected. Given that the model masses were validated in the previous section, a good correlation of modal periods implies that the stiffness properties of the primary SAP2000 model are also consistent.

Table 2: Comparison of the first and second mode periods predicted by the SAP2000 model, Strand7 model and hand calculations using the base springs originally derived at the design stage.

Mode of vibration	Period of vibration, T (s)		
	Hand calculation	SAP2000 model	Strand7 model
N-S Translation	4.5	4.4	4.6
E-W Translation	3.9	3.8	4.0

In order to ensure that the deck response spectra produced in this study is reflective of the dynamic properties of the Maari WHP observed in previous earthquakes, it was agreed with OMV that the original base stiffness would be adjusted in both models such that the first two modal periods of the

structure matched the observed periods. The updated foundation spring stiffness values were calculated by hand followed by an iterative process to 'fine-tune' the springs. In doing so, it was observed that the models matched the observed periods of 3.6s (North-South) and 3.1s (East-West) within $\pm$0.05s.

5 DEVELOPMENT OF DECK RESPONSE SPECTRA

One of the main objectives of this study, as described in Section 1, was to develop elastic response spectra for the future design and assessment of secondary structures at the deck level of the Maari WHP. The steps below were followed in deriving the spectra:

1. A suite of ground motions were selected and scaled appropriately to perform three-dimensional response-history analyses.
2. A response-history analysis was carried out using one of the scaled ground motion records using the primary SAP2000 and the Strand7 models. The deck response from the latter was used to validate the results from the primary model.
3. The response-history analyses using the remaining ground motions were performed in SAP2000 to capture the motion of the deck at discrete locations. The results of the analyses were used to produce acceleration response spectra which were subsequently used to produce smoothed design spectra at the deck level.

A summary of each of the above steps is provided in the following sections.

5.1 Ground motion selection and scaling

The development of deck floor response spectra requires a suite of appropriately selected and scaled earthquake-induced ground motion records to be used as input in performing response-history analysis. Research efforts over the last few decades have resulted in methods for selecting, scaling and/or modification of ground motion to match ‘target’ elastic response spectra in the form of: (i) design response spectra in seismic design codes; or (ii) response spectra of a scenario-based earthquake determined from seismic hazard analysis (refer to Bradley (2012) and references therein). The first method, in particular, forms the basis for selecting and scaling ground motions in the New Zealand loadings standard, NZS 1170.5:2004 (Standards New Zealand, 2004). Despite its limitations, the same approach has been adopted in this study for reasons which will be made clear in the following sections.

5.1.1 *'Target' design spectra*

The process used by Arup to derive so-called ‘foundation design spectra’ is described below because they represent the ‘target’ spectra for scaling the ground motion records in Section 5.1.3

1. Two seismic design levels known as ELE (Extreme Level Earthquake) and ALE (Abnormal Level Earthquake) were considered by Arup according to ISO 19901-2 (International Organization for Standardization, 2004) with target return periods of 200 and 2500 years, respectively. These levels of performance are analogous to the Serviceability Limit State and Ultimate Limit State earthquakes in NZS 1170.5:2004 (Standards New Zealand, 2004). GNS Science (GNS) was subsequently engaged in 2006 to carry out a site-specific seismic hazard assessment (SSHA) for the Maari site to produce bedrock spectra.
2. Arup carried out site response analyses to predict the ground motions in the soil profile extending from the bedrock to mudline (approximately 118m) using: (i) ground motions selected and scaled to match the GNS bedrock spectra; and (ii) a non-linear finite element soil column model.
3. The horizontal motions captured at a depth of 50 m below the mudline were applied to the edges of a smaller/shallower 3D soil model also containing a simplified model of the foundation and superstructure. Motions of the soil captured beneath the foundation were used to produce 'foundation design spectra', which consider how the mass and motion of the structure affects the motion of the soil directly below the foundation.

Figure 3 illustrates the Arup-derived foundation spectrum at the ALE level (2500 years return period) in both horizontal and vertical directions. It should be noted that the vertical spectrum has been obtained by applying a 2/3 scaling factor to the horizontal bedrock spectrum rather than the horizontal mudline

spectrum (which is the approach followed in NZS 1170.5:2004 for onshore structures). This was rationalised at the design stage by the fact that saturated soil is effectively incompressible for vertical motions. Also shown for comparison in Figure 3 are response spectra obtained using NS1170.5:2004 and the GNS SSHA for the Maari site assuming site subsoil class D conditions. The following observations can be made from this comparison:

- The short-period ($T \leq 0.5$ s) spectral accelerations in the Arup-derived horizontal foundation spectrum are significantly lower than the NZS 1170.5 and GNS spectra. This can be explained by the mass and motion of the WHP acting to filter out the high frequency motion in the soil.
- At longer periods ($T \geq 1.5$ s), the differences between the Arup and GNS spectra are minimal, which implies that the Arup approach does not offer reduction in the overall horizontal seismic loads at the primary modes of vibration.
- The vertical foundation design spectrum exhibits significantly lower spectral accelerations in the period range corresponding to the vertical modes of the WHP (0.4s $\leq T \leq$ 1.0s) when compared to the NZS 1170.5 spectrum. Hence, significant differences in the vertical response of the platform can be expected depending on the chosen spectrum.

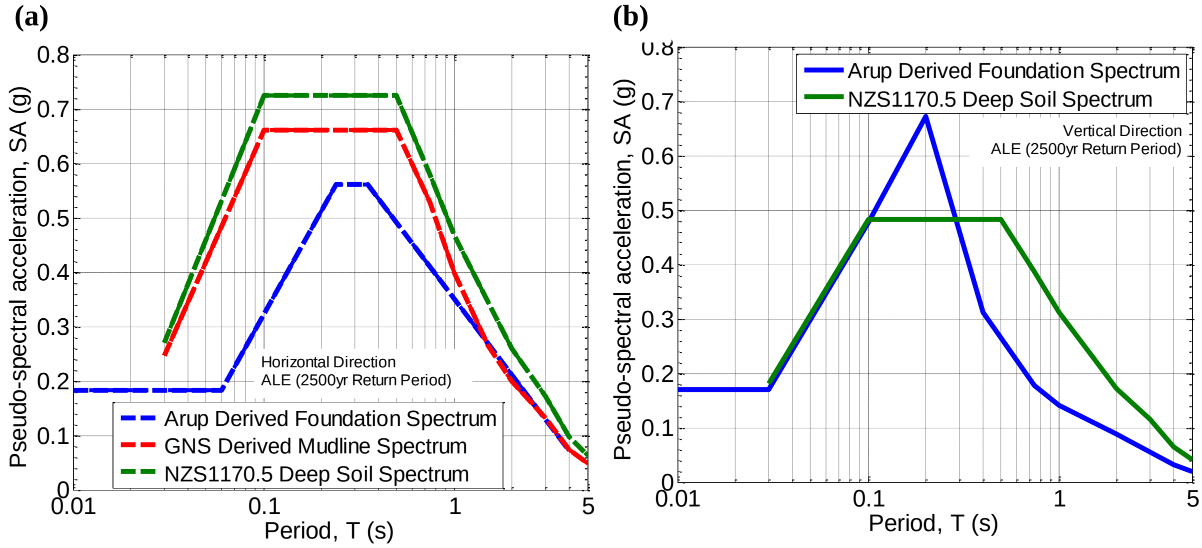

Figure 3: Comparison of the foundation design spectrum derived by Arup for the Maari WHP site with spectra derived from the GNS Science site-specific seismic hazard assessment (horizontal direction only) and NZS 1170.5:2004 for site subsoil class D conditions in the: (a) horizontal; and (b) vertical directions.

5.1.2 *Consideration of the generalised conditional intensity measure approach*

The main limitation of the aforementioned ground motion selection methods, as stated by Bradley (2012), "stems from the fact that only characteristics of the ground motion represented in terms of (elastic) spectral accelerations are considered, whereas it is well acknowledged that the severity of a ground motion, in general, depends on its intensity, frequency content, and duration".

Upon recognising the limitations associated with these conventional methods, Bradley (2012, 2010) developed the GCIM (Generalized Conditional Intensity Measure) approach, which is a methodology for ground-motion selection that is consistent with the results of site-specific seismic hazard assessment. In essence, the GCIM approach is a framework in which the distribution of any ground motion intensity measure can be obtained given the occurrence of another specific ground motion intensity measure (Bradley, 2012). It therefore allows any number of intensity measures which may be considered to be important to the seismic response analysis. The resulting GCIM distributions, which are treated as the 'target', are subsequently used in an algorithm which carries out the ground motion selection in a holistic manner. This is based on statistical goodness-of-fit tests between the empirical distribution of the ground motion suite and the theoretical distribution of a particular intensity measure obtained from the GCIM approach. The details of the algorithm are omitted herein as they are outside the scope of this paper and can be found in Bradley (2012, 2010).

The GCIM approach for ground motion selection was initially considered in this study because it: (i) addresses the primary limitation of traditional ground motion selection methods describe above; and (ii) allows selection of ground motions to occur in a manner that does not require the user to exercise judgement and is consistent with the expected seismic hazard at the site. However, the use of this method was not progressed further for the following reasons:

- The 'target' for ground motion selection and scaling (i.e., GCIM distributions) is not consistent with the original basis of design response spectra for the WHP discussed in Section 5.1.1.
- For amplitude-based scaling of ground motions (considered in this study), the GCIM-selected motions are scaled to the chosen conditioning intensity measure (e.g., spectral acceleration at the first mode period of the structure, $SA(T^*)$). This may not be appropriate for predicting the response of secondary structures at the deck whose periods are likely to vary significantly (based on information received from OMV).

5.1.3 *Consideration of the NZS 1170.5:2004 approach*

NZS 1170.5:2004 (Standards New Zealand, 2004) requires ground motion records to be selected such that they have a seismological signature (i.e., magnitude, faulting mechanism and source-to-site distance) which is reasonably consistent with the signature of events contributing to the target design spectra over the period range of interest. In addition, the standard requires the ground motions to have been recorded by instruments at sites with similar sub-soil conditions to the site under question.

The NZS 1170.5 approach was chosen in the present study primarily because it allows amplitude scaling of ground motions to occur over a wide range of periods, which in this case corresponds to the vibration periods associated with appurtenance structures at the deck level. The input ground motion selection process was guided by the results of an extensive study carried out by Oyarzo-Vera et al. (2012). The aim of their study was to carry out seismic hazard zonation of the North Island and select suites of ground motion records for each zone in accordance with the criteria above. The Maari WHP is located in Zone A, which according to Oyarzo-Vera et al. (2012), can be associated with a large north-western area of the North Island where the 500-year return period hazard is governed by distributed seismicity sources rather than specific faults. These sources are characterised as normal-faulting earthquakes in the most recent update to the New Zealand seismicity model (Stirling et al., 2012). The above findings are consistent with SSHA carried out by GNS and can be extended to the 2500-year return period (ALE) hazard for the periods of interest. The contribution of the Cape Egmont Fault (located approximately 10 km from the site) to the hazard is most notable only at longer return periods and for vibration periods greater than T = 1.0s.

A suite of seven ground motion records selected by Oyarzo-Vera et al. (2012) for deep soil conditions (site subsoil class D) were used for the response-history analysis of the WHP. It is important to note that these records have been obtained from regions of extensional tectonic regimes, and correspond to earthquakes with moment magnitudes of $5.5 \leq M_w \leq 7.0$ and source-to-site distances of 10-30 km. These records were subsequently scaled to the 'target' design spectra (refer to Section 5.1.1) using the NZS 1170.5:2004 approach, based on least-squares fit over the period range, $(0.4T_1, 1.3T_1)$, where T_1 represents the fundamental elastic period of the structure. Based on information received from OMV regarding likely vibration periods of deck appurtenance structures, it was considered appropriate to scale the records between T = 0.4s and T = 1.3s. The criterion requiring the record scale factor (k_1) to be between $0.33 < k_1 < 3.0$ was satisfied by all records. Figure 4 illustrates the response spectra of the scaled ground motions and the design spectra developed by Arup for the two horizontal directions. The average of the seven scaled response spectra corresponding to each component is also shown for comparison with the design spectrum. In the absence of information on scaling the vertical component (not shown) in the standard, the scaling factor applied to the horizontal components was adopted based on guidance provided by the NIST (NEHRP Consultants Joint Venture, 2011) research document.

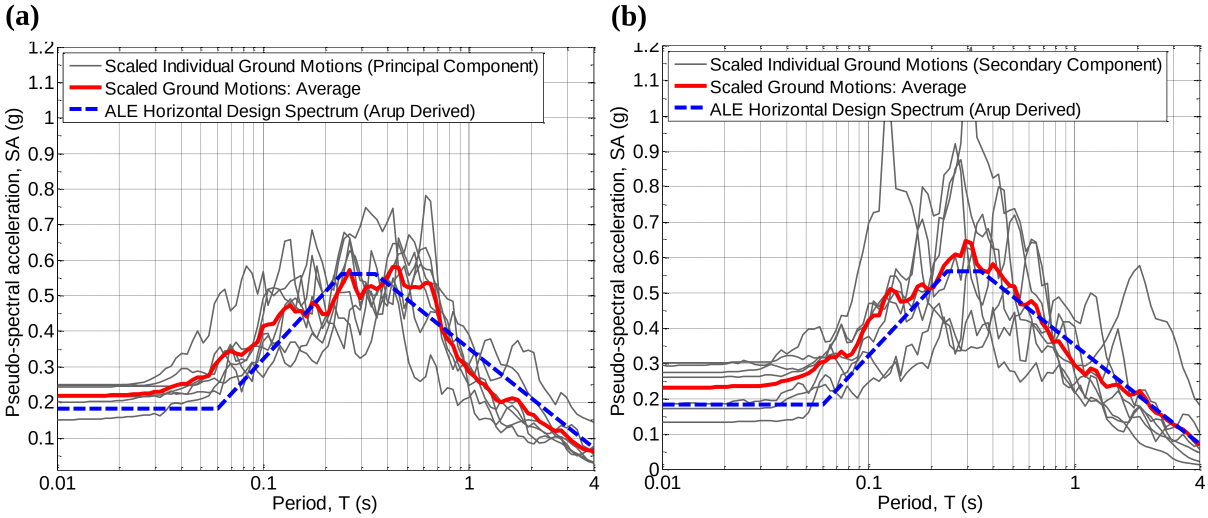

Figure 4: Response spectra of ground motions scaled to the 'target' design spectrum derived for the Maari WHP in accordance with NZS 1170.5:2004 for the: (a) principal; and (b) secondary horizontal components.

5.2 Response-history analyses

A series of three-dimensional response-history analyses were carried out to predict the WHP response by applying the scaled ground motions selected in the previous section to the base of the SAP2000 analytical model. The analyses have been based on a number of assumptions that we consider to be reasonable:

- The response of the WHP, for the purposes of deriving deck response spectra, is linear elastic for the earthquake levels considered. This is expected to produce conservative results for the design spectra.
- Damping levels of 5% are assumed for the WHP and appurtenance/secondary structures.
- The deck floor response spectra have been developed for the current ALE (Abnormal Level Earthquake) basis of design. A scaling factor known as the reserve capacity factor (C_r), is applied to obtain the response spectra for the ELE (Extreme Level Earthquake) case, in accordance with ISO 19901-2 (ISO, 2004).

5.3 Validation of response-history analysis results

The validation of the response-history analysis results was carried out by comparing: (i) displacement response histories; (ii) pseudo-acceleration response spectra at the deck level of the primary WHP SAP2000 and validation Strand7 models for one input ground motion record (El Centro, 1940 M_w 7.0 Imperial Valley Earthquake).

Figure 5a and Figure 5b illustrate the horizontal (East-West only) and vertical displacement response of the deck to the El Centro ground motion record from both models. It can be observed that there is good agreement between the responses predicted by both models.

A comparison of the deck acceleration response spectra in the horizontal (East-West only) and vertical directions for the same ground motion obtained using both models is illustrated in Figure 6. The plots show general agreement between the models with the SAP2000 model producing higher response across a majority of the vibration periods. The differences in the horizontal spectral accelerations in the 1s $\leq T \leq$ 2s range can be attributed to differences in the primary torsional periods. We believe that these differences have arisen due to variations in the modelling of the deck-to-jacket connection between the two models. Similar trends were also observed in the North-South direction. The differences were not explored further as they were considered to be within reasonable bounds for the purposes of this study, and also because the SAP2000 model provided generally conservative results compared to the Strand7 model.

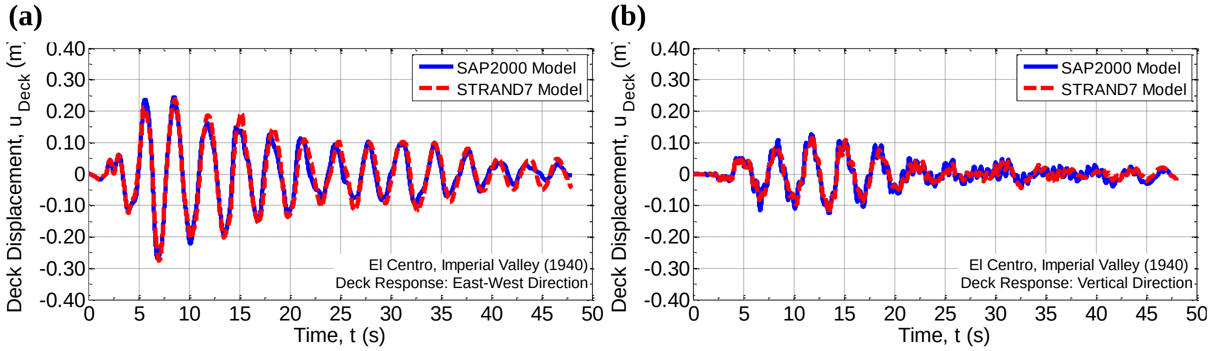

Figure 5: Comparison of the WHP deck displacement response to the El Centro ground motion record obtained using the primary SAP2000 and validation Strand7 models in the: (a) horizontal (East-West); and (b) vertical directions.

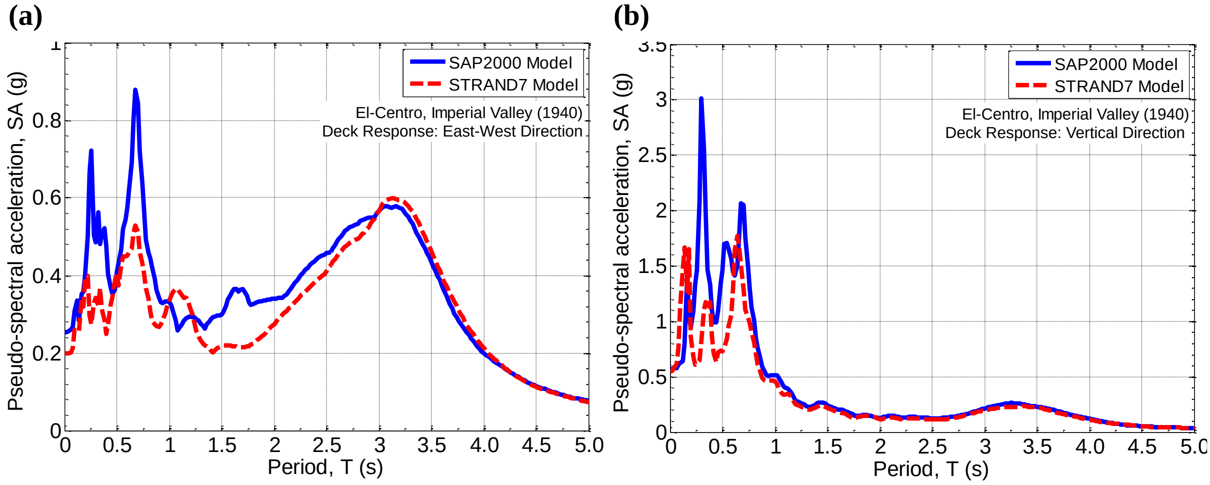

Figure 6: Comparison of the WHP deck pseudo-acceleration response spectra for the El Centro ground motion record obtained using the primary SAP2000 and validation Strand7 models in the: (a) horizontal (East-West); and (b) vertical directions.

5.4 Features of the recommended deck response spectra

The response of the deck to each of the seven ground motion records was captured at each of the four deck corners because: (i) they are expected to provide the worst-case deck accelerations for both horizontal and vertical directions; and (ii) there is significant torsional response due to the eccentricity of deck in plan with respect to the supporting jacket.

Figure 7 and Figure 8 illustrate the raw and recommended ALE spectra (shown in grey and blue, respectively) for deck motions in the horizontal and vertical directions. The average of the raw spectra has been shown in black. It should be noted that only the response spectra in the North-South direction has been shown as the East-West spectra exhibited similar features. In order to understand the features of the raw spectra, the peak spectral ordinates were initially identified and correlated to the corresponding vibration modes, as illustrated in both figures. The recommended horizontal and vertical seismic design spectra at the deck level are, for simplicity of use, presented as a series of straight lines judged to be a prudent 'best fit' (rather than an envelope) of the spectra derived for individual earthquakes at the four deck locations.

The following key features have been included in the recommended horizontal ALE design spectra in Figure 7 based on observations from the raw spectra obtained from the response-history analyses:

- A short plateau is provided in the period range, 0s $\leq T \leq$ 0.2s to allow for very stiff appurtenance structures that can be approximated as rigid. This is consistent with the guidance provided in the API R2PA (American Petroleum Institute, 2000) document.

- A plateau from $0.25\text{s} \leq T \leq 0.75\text{s}$ represents the periods range for which the peak response can be expected. This period range coincides with: (i) deck mode(s); and (ii) second translational and torsional modes which are excited most by earthquake-induced ground motions.
- There are no significant modes associated with the period range, $1\text{s} \leq T \leq 2\text{s}$, except for the first torsional mode which shows a small local peak in the response at approximately $T = 1.7\text{s}$.
- The period range, $2.5\text{s} \leq T \leq 4.0\text{s}$, represents excitation due to the primary translational mode of the structure.

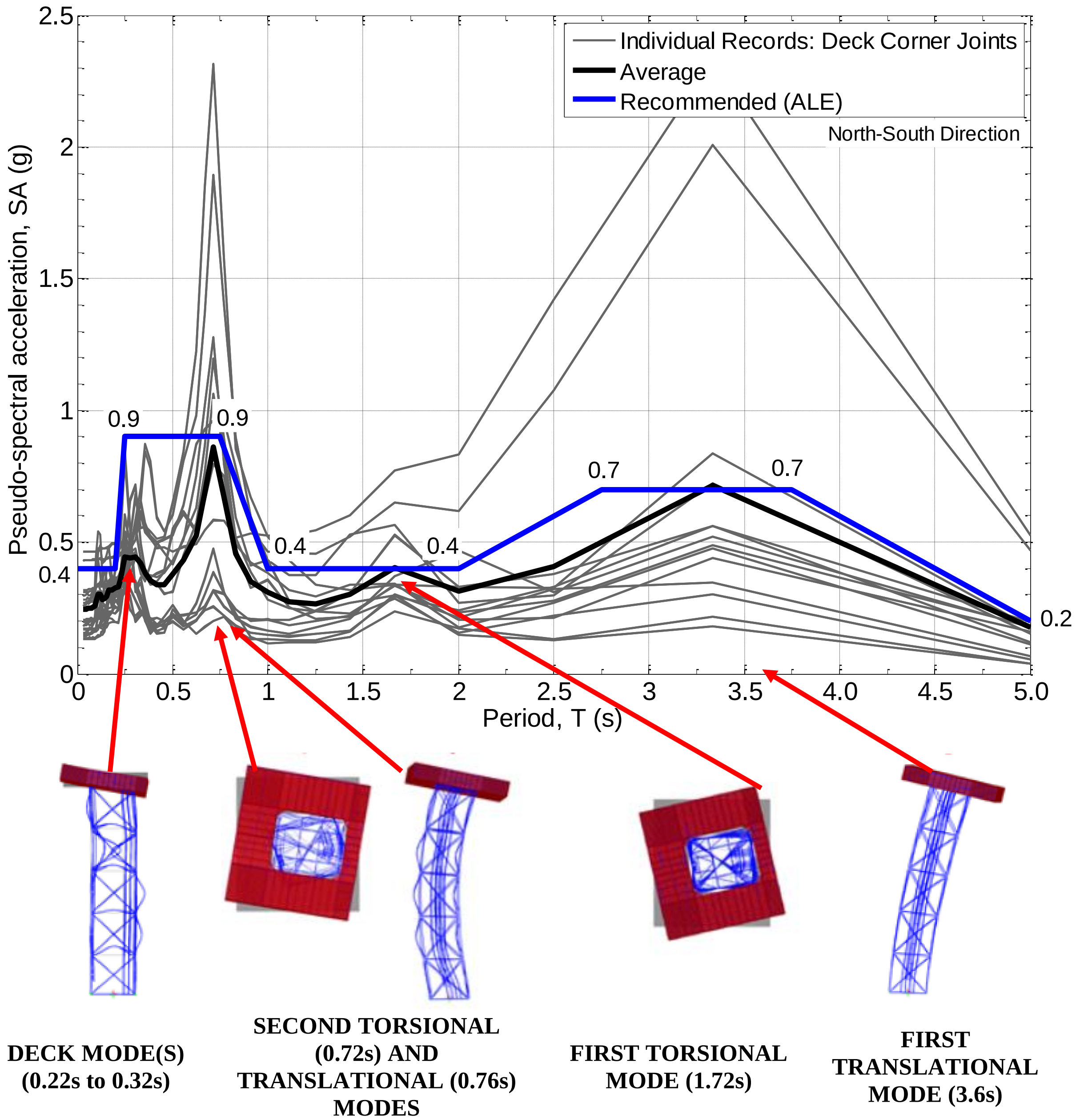

Figure 7: Recommended horizontal design response spectra at the deck level of the Maari WHP developed using the raw spectra obtained from response-history analyses. The mode shapes associated with the peaks of the raw spectra are also shown for reference.

The following key features have been included in the recommended vertical design spectra in Figure 8 based on observations from the raw spectra obtained from the response-history analyses:

- Similar to the horizontal spectra, a short plateau is provided in the period range, $0\text{s} \leq T \leq 0.2\text{s}$. Most normal structures are very stiff in the vertical direction, and for this reason it is expected

that most appurtenance structures fit into this period range. The exceptions to this rule, for example, may be cantilevered structures such as cranes or overhanging platforms.

- The peak response can be expected between $0.2\text{s} \leq T \leq 0.75\text{s}$, which coincides with deck modes that have a significant vertical component, particularly at the corners of the deck. Hence, structures with vertical modes in this period range require careful consideration.
- Beyond $T = 1\text{s}$, the expected vertical excitation is not significant relative to the short period range. This implies that structures that are very flexible in the vertical direction are effectively isolated from the vertical deck motion, which contains mainly high-frequency content.

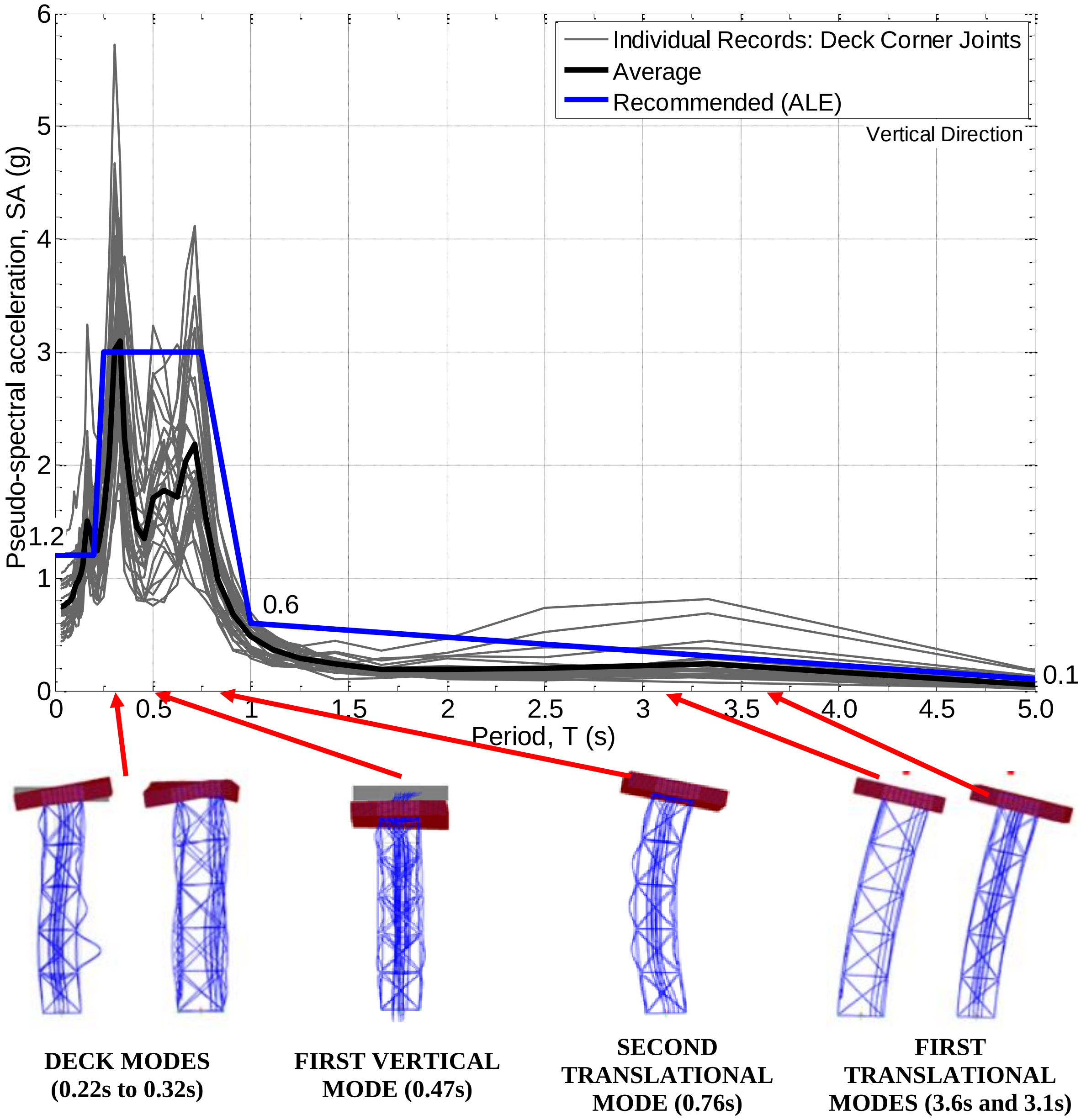

Figure 8: Recommended vertical design response spectra at the deck level of the Maari WHP developed using the raw spectra obtained from response-history analyses. The mode shapes associated with the peaks of the raw spectra are also shown for reference.

6 CONCLUSIONS

The presence of accelerometers at the deck level of the Maari Wellhead Platform (WHP) provided an excellent opportunity to understand the dynamic behaviour of the structure when subjected to significant ground shaking caused by seismic events such as the 21 July 2013 $M_w6.5$ earthquake (located approximately 200 km south-east from the WHP site). More importantly, observations from this event

highlighted the fact that the fundamental vibration periods of the structure are approximately 30% less than those estimated during the original design of the structure. In an effort to understand this discrepancy, OMV (owner and operator of the platform) engaged Beca to develop a detailed analytical model of the structure and investigate its natural frequencies. The model was developed in the SAP2000 structural analysis package using up-to-date operating mass information and foundation springs derived during the design stage. The larger periods predicted by the Beca model and models developed during design stage were primarily attributed to the foundation springs being too flexible for transient-type loading.

Based on the findings from the initial stage of this study, OMV engaged Beca to develop seismic design response spectra for future design and assessment of appurtenance structures located at the deck level of the Maari WHP. In order to do so, the SAP2000 model was firstly calibrated such that consistency between the predicted and observed translational modal periods was achieved. The dynamic properties of this model were validated thoroughly by means of a validation model developed from 'as-built' drawings of the WHP using the Strand7 finite element software. Deck motions were predicted by performing response-history analyses of the WHP using a suite of earthquake-induced ground motions. Although a state-of-the art methodology known as the GCIM (Generalized Conditional Intensity Measure) approach (Bradley, 2012, 2010) was initially considered, a set of seven ground motions selected by Oyarzo-Vera et al. (2012) as part of a seismic hazard zonation study of the North Island were adopted in this study and scaled in accordance with NZS 1170.5:2004. The conventional amplitude-based scaling method in the standard was preferred as it allowed the ground motions to be scaled to the basis-of-design response spectra over a wide range of periods, which in this case correspond to the likely vibration periods of appurtenance structures.

The results of the response-history analyses obtained using the primary SAP2000 model were validated using the Strand7 model by comparing the displacement response histories and deck acceleration response spectra for one ground motion record. Recommended seismic design response spectra in both horizontal and vertical directions at the deck level were derived as a series of straight lines using engineering judgment to reflect the key features of the spectra obtained from the analyses at a number of deck locations. These spectra are likely to be of particular use in the future design and assessment of appurtenance structures that cannot be treated as rigid bodies, and exhibit their own dynamic behaviour to the deck motions.

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