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SEISMIC DESIGN OF STONECUTTERS BRIDGE, HONG KONG

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ABSTRACT :
Stonecutters Bridge will have a main span of 1018m to cross the Rambler Channel in Hong Kong, China. The 1596m long cable-stayed bridge has a steel main span and concrete back spans. The tapered mono column towers stand on the bridge centre line between the twin deck box girders, which are connected with cross girders. The towers are concrete to +175m and composite with an outer stainless steel skin to +293m, topped by a lighting feature to +298m. Stay cables are in 2 planes in a modified fan arrangement attached to the outside edges of the deck girders. The concrete back span decks are monolithic with the three intermediate single shaft piers and the portal end piers. Both sides of the bridge are on reclaimed land and foundations are large diameter bored piles to bedrock.

Short span bridges in Hong Kong are designed for a 5%g nominal lateral earthquake load. For long period structures design response spectra are required. For Stonecutters Bridge, these were developed for various return periods using a Probabilistic Seismic Hazard Assessment (PSHA). The effects of the local soil profile were also explicitly considered. The paper summarizes the PSHA and how the effects of the local soil profile were taken into account using a one dimensional non-linear dynamic site response analysis to determine the design spectra at ground level. The methods used to determine the dynamic response of the bridge and the effects of non-synchronous motion of the foundations are also presented.

KEYWORDS: Bridge design, Hong Kong, Design spectra

1. INTRODUCTION

While there are currently no seismic design requirements for buildings in Hong Kong, civil engineering structures, including bridges, are required to be design to resist seismic loading. For short span bridges this results in a nominal lateral load of 5%g applied to the dead load. For large span bridges however this becomes an unrealistic requirement and it is necessary to show that a range of performance requirements are met at various levels of seismic ground motion. This paper introduces how these requirements have been developed for Stonecutters Bridge that is currently under construction in Hong Kong. It goes on to describe how the seismic loading was generated for the bridge and what affect the seismic loading had on the design of the elements of the bridge. The likelihood and effects of liquefaction and non-synchronous ground motion are also presented.

2. BRIDGE DESCRIPTION

Stonecutters Bridge, currently under construction, is a cable-stayed bridge with a main span of 1018m as shown in Figure 1. The high level deck is steel in the main span and the first 49.75m of each back span. The remainder of the back spans are prestressed concrete, making the total length of the bridge 1596m. The 295m tall circular tapered mono-column concrete towers stand on land at the bridge centre-line, between the two longitudinal boxes of the twin girder deck. Stay cables are in 2 planes arranged in a modified fan layout and attached to the outside edges of the deck. The girders are connected with cross girders spaced at 18m in the main span, coinciding with the stay anchorage spacing.

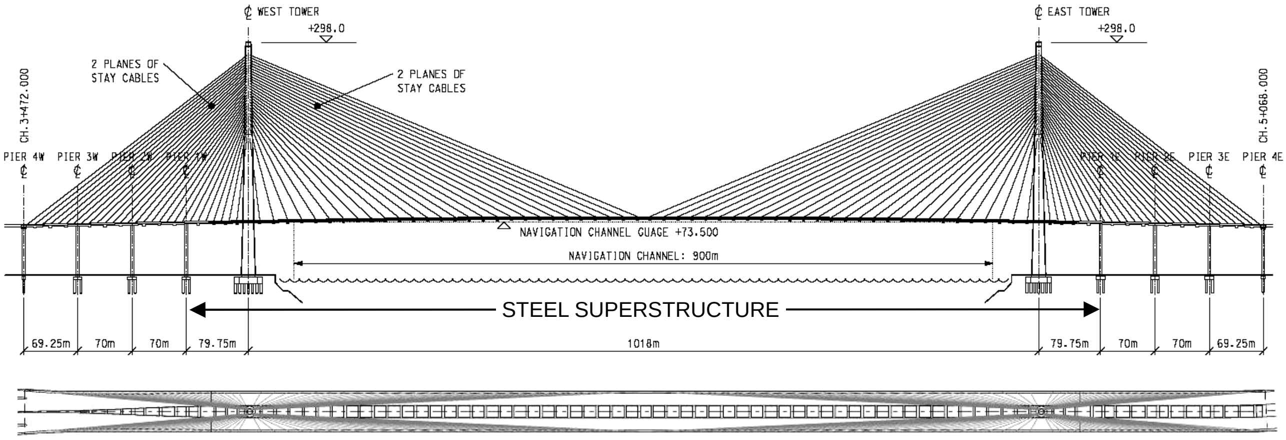


Figure 1 Elevation and plan

3. DESIGN REQUIREMENTS

Three limit states were considered in the design as follows:

Serviceability Limit State (SLS): The bridge shall behave elastically and is expected to be serviceable immediately without the need for any repair. For seismic load-cases, this condition will be applicable to frequently occurring or minor earthquakes with a return period of 120 years. General damage level is *no-damage.*

Ultimate Limit State (ULS): The bridge may undergo large deformation in the post-elastic range without substantial reduction in strength and limit the damage level of the bridge to that which is economically and technically feasible to repair. For seismic load-cases, this condition will be applicable to a moderate earthquake with a return period of 2400 years. General damage level is *minimal.*

Structural Integrity Limit State (SILS): The deformation and damage of the bridge shall not be such as to endanger emergency traffic or cause loss of structural integrity. For seismic load-cases, this condition will be applicable to a severe earthquake with a return period of 6000 years. General damage level is *repairable.*

3.1 Performance Level Definitions

The performance levels expressed in terms of damage levels are as follows:

No Damage: Defined for structural members as the nominal capacity at service load level as per the BS5400.

Minimal Damage: Essentially elastic performance. Although minor inelastic response may occur, post earthquake damage shall be limited to narrow cracking in concrete and inconsequential yielding of secondary steel members. Damage to non-structural components would be allowed.

Repairable Damage: Inelastic response may occur, resulting in concrete cracking, reinforcement yielding, minor spalling of cover concrete, and minor yielding of structural steel. The extent of damage should be sufficiently limited that the structure can be restored essentially to its pre-earthquake condition without replacement of reinforcement or replacement of structural members. Repair should not require bridge closure.

Significant Damage: No collapse, but major damage may occur. Damage may consist of concrete cracking, reinforcement yielding, major spalling of concrete, yielding of structural steel and deformation in minor bridge components. Repair would require bridge closure.

4. SEISMIC HAZARD

The conventional probabilistic seismic hazard assessment (PSHA) methodology, e.g. Cornell (1968), McGuire (1993), has been applied using *Oasys* SISMIC, the in-house PSHA program of Arup. The use of this seismic hazard assessment methodology to estimate the potential seismic ground motion levels on bedrock in Hong Kong has been previously published in Free et al. (2004). Epistemic uncertainty arising from differences in expert opinion on a range of modeling assumptions has been addressed through the use of a logic tree (Coppersmith and Youngs, 1986). Aleatory uncertainty, arising from natural physical variability, has been addressed by allowing for the normal variation, represented by its standard deviation, of the ground motion attenuation relationships in the hazard computation. The attenuation relationships used in the hazard analysis are presented in Free et al. (2004).

4.1 *Seismological Data*

Historical earthquake data for the Southeast China region (Guangdong Province of China) has been obtained from a range of sources. The Directory of Earthquakes in China (BC 1831 to AD 1969) as listed in Gu et al. (1983) and the Guangdong Seismological Bureau (2002) database provide the most extensive catalogues of instrumentally recorded earthquake data for the region. These have been supplemented by the data from the GCO (1991).

For the PSHA, all events are required to be statistically independent and therefore foreshocks and aftershocks have been removed from the catalogue using the methodology of Gardener and Knopoff (1974). Man-induced events have also been identified and removed from the catalogue. A reservoir induced earthquake swarm commenced in 1962 during the filling of the Xinfengjiang Reservoir.

Earthquake recurrence curves for events within a distance of 500km from Hong Kong for the onshore and near-shore offshore zones and for the offshore zones are shown in Figure 2. For the onshore and near-shore seismic source zones three completeness ranges have been defined (since 1500 for M≥7.0, since 1870 for M≥5.0 and since 1970 for M≥2.5). For the offshore seismic source zones two completeness ranges have been defined (since 1920 for M≥5.5 and since 1971 for M≥4.5). The seismic source zones used in the PSHA are shown in Figure 3. The PHSA for Stonecutters Bridge differed from that presented in Free et al. (2004) in that this was the only source model considered.

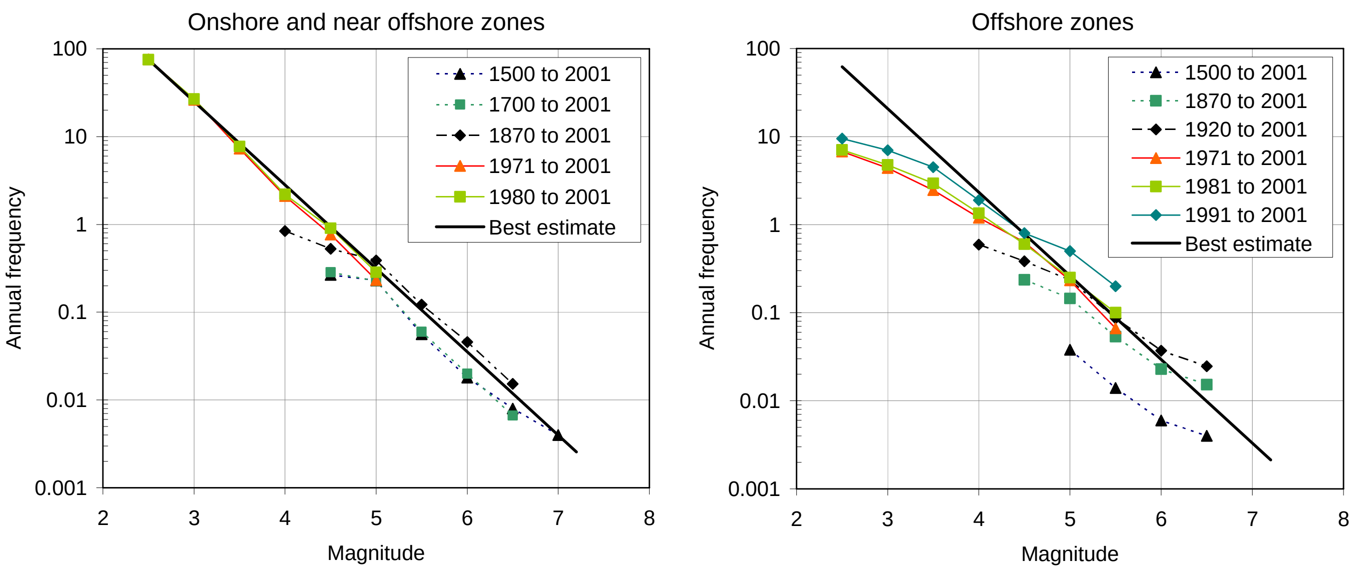


Figure 2 Recurrence plots for events within 500km of Hong Kong

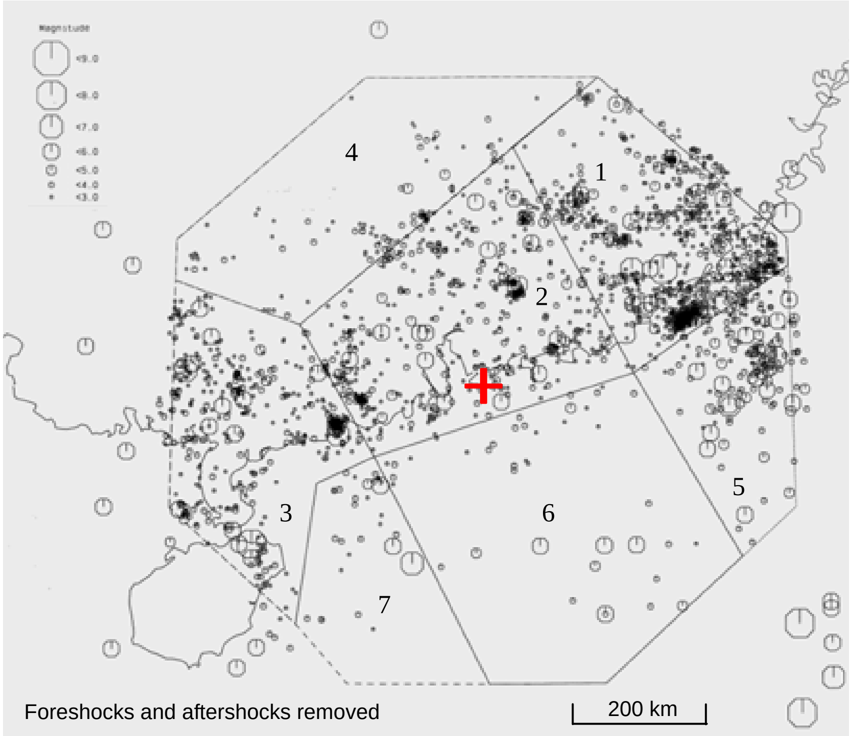


Figure 3 Seismic source model adopted for the seismic hazard assessment

4.3. Site response

The ground conditions at the site comprise about 30m of loose sand fill, over 15m of dense alluvium over 10m of completely weathered granite over bedrock. *Oasys* SIREN, the Arup in-house program for analysis of the response of a 1-dimensional soil column subjected to an earthquake motion at its base, was used to determine the seismic ground motion at the site. Detailed descriptions and calibrations of the program SIREN are presented by Henderson et al. (1990) and Heidebrecht et al. (1990). Theoretically, the soil model used in SIREN more accurately reflects actual hysteretic soil behaviour when compared to pseudo-non-linear soil models used in many other site response programs (e.g. SHAKE, Schnabel et al. ,1972). The small strain shear modulus (Go) profile at the site was determined from insitu down-hole shear wave testing carried out using a very large pendulum hammer striking a wooden plank under a large concrete block (see Figure 4). The G/Go relationships with shear strain amplitude have been presented previously in Pappin et al. (2004).

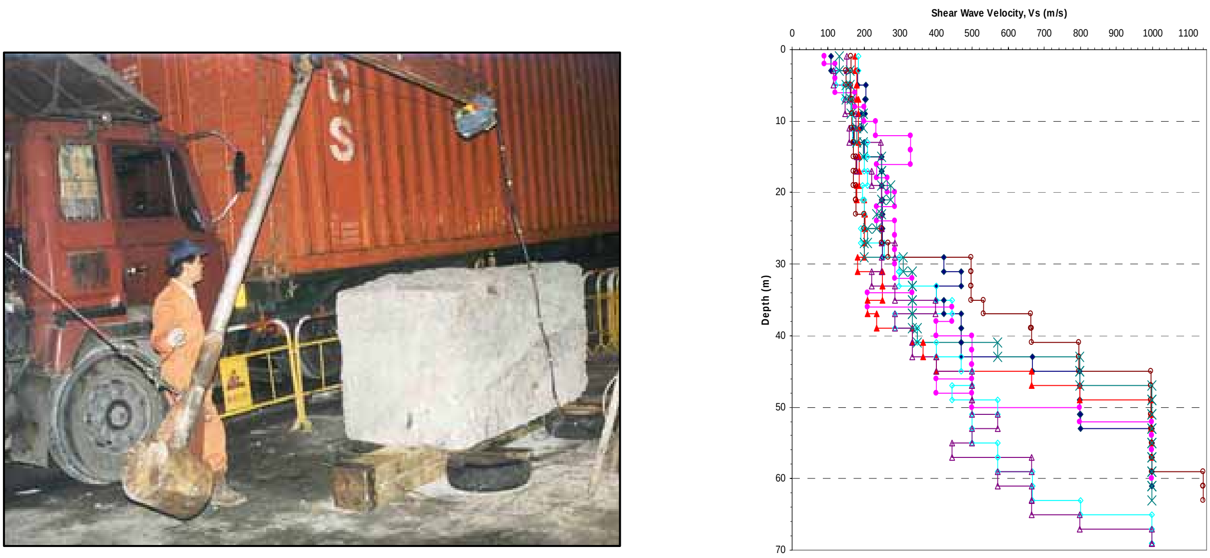


Figure 4 Insitu shear wave testing and derived Go profiles

4.4. Calculated Seismic Hazard

The calculated seismic hazard at bedrock (shown by the squares) and at the soil surface are presented in Figure 5 for return periods of 120 and 6000 years. The 2400 year return period curves are about 70% of the 6000 year values. The straight line envelope curves used in the design are also shown.

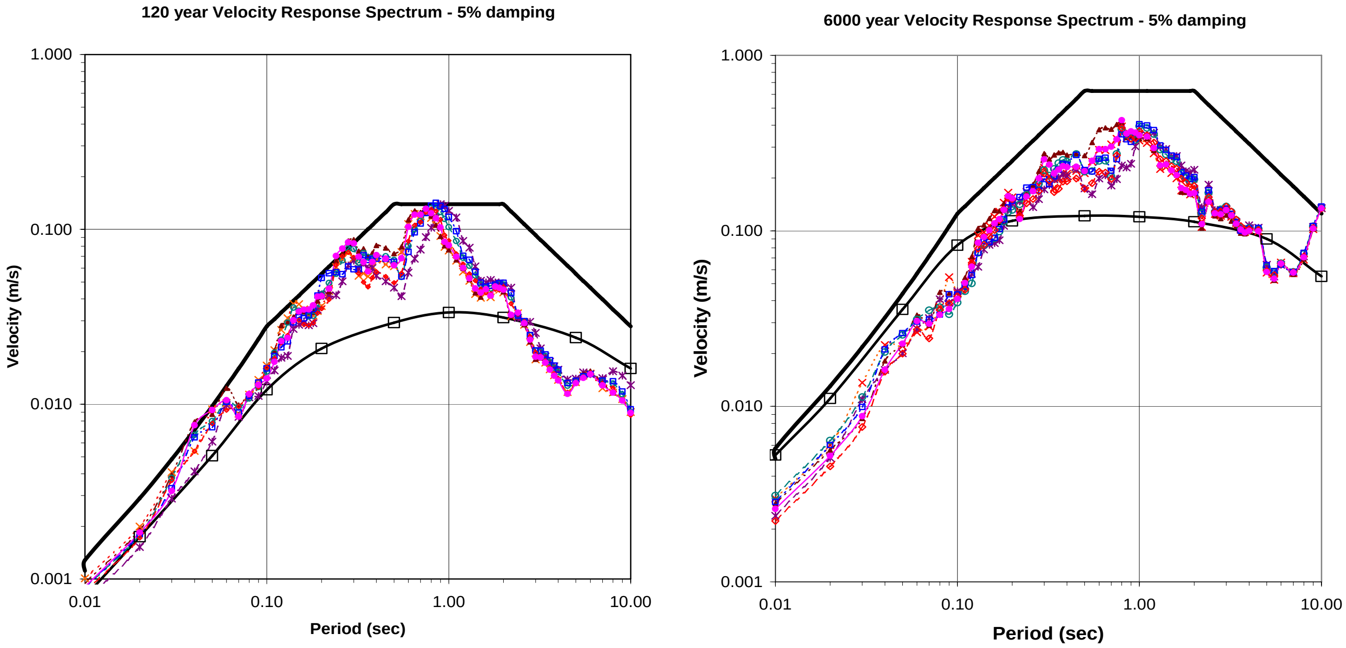


Figure 5 Calculated bedrock and soil surface response spectra

4.5 Liquefaction

The probability of liquefaction was studied and considered to be likely at SILS. Lower values of soil shear modulus over the affected depth where liquefaction may occur were considered in the pile group analysis for the SILS condition.

5. SEISMIC EFFECTS ON THE BRIDGE

5.1 Analysis

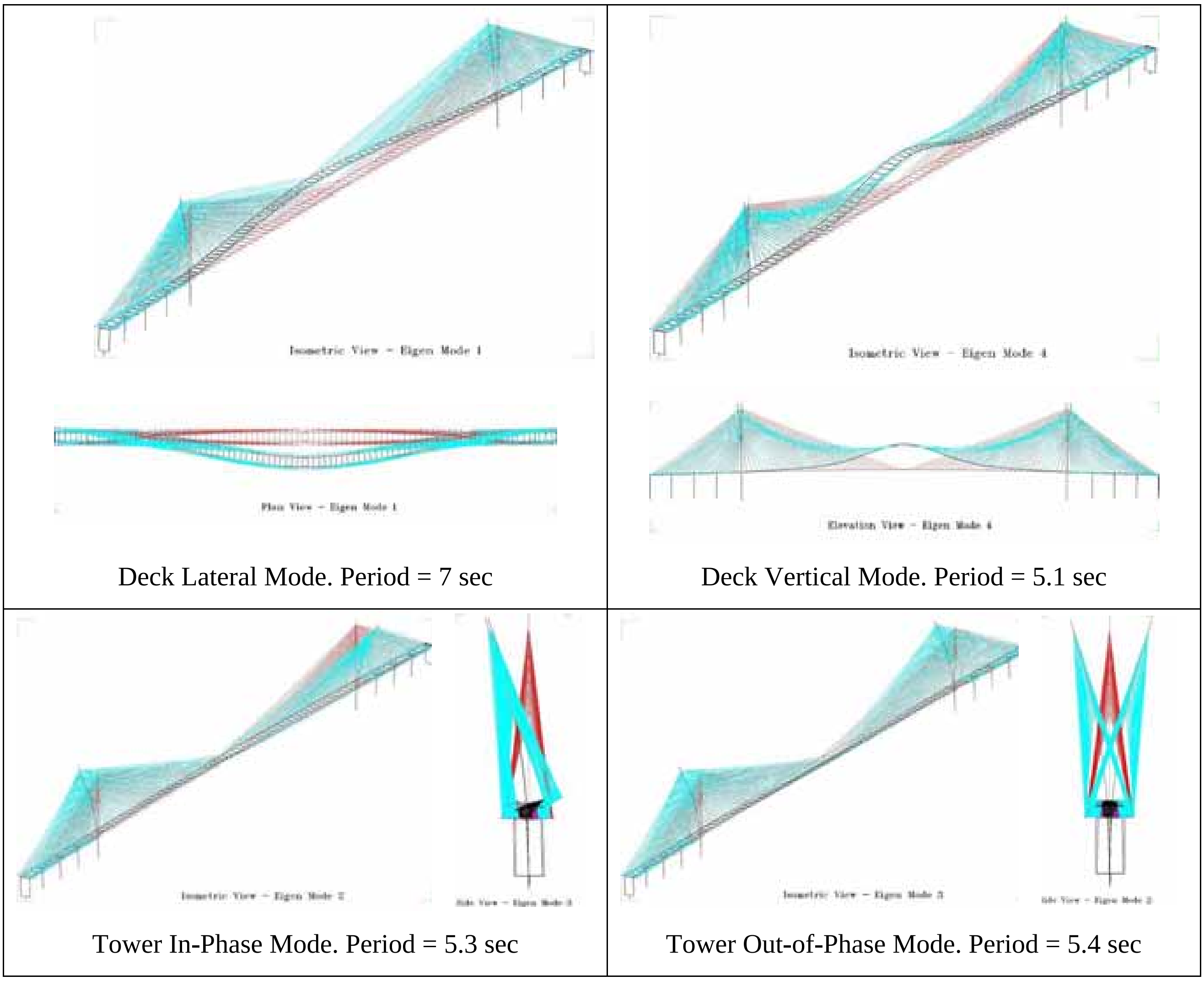
The global analysis of the bridge structure was performed using a space frame model. The foundations were represented by springs at the underside of pile-cap level, with stiffnesses determined from the geotechnical pile group analyses taking into account the ground conditions. Dynamic response spectral analysis was used to assess the effect of earthquake loading on the bridge structure and foundations. The design spectra for the three limit states (serviceability, ultimate, and structural integrity) were applied to the dynamic analysis model. Each limit state analysis was performed in three excitation directions, using the vertical spectra, and the horizontal spectra for both the longitudinal and transverse directions. This determined the interaction between the frequencies of ground motion and the natural frequencies of the structure. The total mass included in the analysis was all permanent vertical loads and one third of type HA traffic loading on one lane in each direction.

The CQC (Complete Quadratic Combination) was used for combination of the effects from different modes. While the CQC method is reasonable for single action effects (one excitation direction only), it is difficult to apply to multiple action effects arising from different excitation directions that interact with each other. For the combination of excitation directions a 100:40:40 combination rule was used, with 100% contribution the primary direction and a 40% combination from the other two directions. Each direction was taken in turn as the primary direction, and all results were enveloped to determine the worst load effects.

In cases where liquefaction of the soil was assumed to have occurred, the foundation spring stiffnesses were modified based on updated results from the geotechnical pile group analyses. The spectral analyses were then re-run for these cases to ensure any changes in modal response due to liquefaction were taken into account.

5.2 Primary mode shapes and Frequencies

The primary mode shapes of the structure are for lateral bending of the deck, transverse bending of the towers (both in- and out-of-phase), and vertical bending of the deck. The periods for each of these primary modes range from 5.2 seconds to 7 seconds. The first 300 modes were taken into account in the seismic spectral analysis. Figure 6 shows a selection of the modes of vibration.



Deck Lateral Mode. Period = 7 sec

Deck Vertical Mode. Period = 5.1 sec

Tower In-Phase Mode. Period = 5.3 sec

Tower Out-of-Phase Mode. Period = 5.4 sec

Figure 6 Natural Modes of Vibration

5.3 Non-synchronous Ground Motion

Given the 1km main span the relative movement between supports was assessed by examining a displacement time history analysis to determine the maximum out of phase differential horizontal displacements that may occur assuming a seismic wave propagation velocity of 1000m/s. The resulting change in lateral movement between the supports are listed below in Table 1. Simple static rules were then be used to add this effect into the results from the standard spectral analysis.

Table 1 Difference in base movement due to non-synchronous motion

Design Earthquake Level	**Tower to Tower (mm)**	**Backspan to Backspan (mm)**
Serviceability Limit State (120 years)	±5	±2
Ultimate Limit State (2400 years)	±19	±6
Structural Integrity Limit State (6000 years)	±25	±9

5.4 Foundation Design

A section through the East Tower and Back Spans showing the ground conditions and the pile foundations is shown in Figure 7. There are 29 piles, each 2.8m diameter in the group under the tower and 6 piles in the group under each of the piers, typically 2.5m diameter.

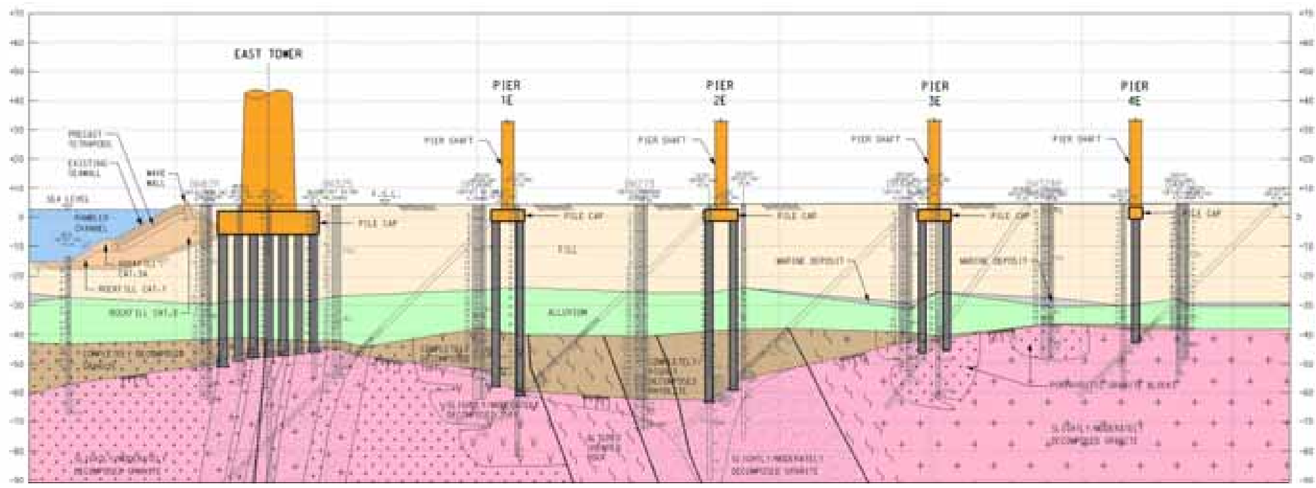


Figure 7 Section through the East Tower and Back Spans

During an earthquake the ground around the pile group will experience horizontal and vertical translational movements that will have an effect on the pile foundations. The effect of this relative movement was determined using a finite element analysis program with two-dimensional slices being taken through each of the two major axes of each foundation group. The piles and pile cap were represented in the model as a series of elements with the appropriate concrete material properties. The piles were were connected to the soil mesh using interface elements that allowed relative movement between the piles and the surrounding soil. The effect of the ground movement on the piles was modeled by inducing the soil displacement profiles derived from the site response analysis as a pseudo static load case. Figure 8 schematically shows the resulting pile movement for a typical seismic load case.

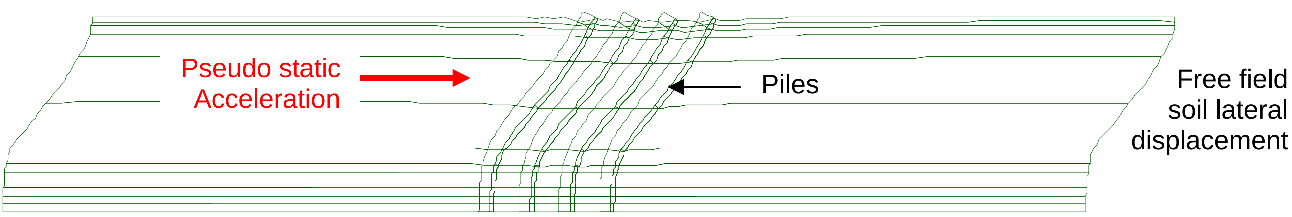


Figure 8 Pile movements below the soil surface

For design of the piles, the two loading effects of global forces from the structural analysis and the effects of field ground displacements were considered together. The bending moments, shear forces and translation of the piles determined from the field displacement analysis were superimposed with the values determined from the pile group static analyses for each of the respective seismic conditions. A combination rule which took the maximum effect in one direction, and 40% of the maximum effects in all other directions as combined effects was adopted.

5.5 Bridge Design

The seismic forces in the bridge structure calculated by the response spectral analysis have a more complicated distribution than forces calculated by a simple equivalent lateral load analysis. However, it is worth noting that for the towers, similar foundation forces can be calculated by laterally applying the proportions of gravity listed in Table 2.

Table 2 Equivalent lateral loads

Design Earthquake Level	Equivalent Lateral Load to generate Tower Foundation Loads
Serviceability Limit State (120 years)	0.016 g
Ultimate Limit State (2400 years)	0.046 g
Structural Integrity Limit State (6000 years)	0.072 g

However, in the back spans shear forces are underestimated by this equivalent loading by a factor of up to 3, and the bending moments are overestimated by a factor of up to 1.5. Hence the method of applying a proportion of gravity laterally is an over-simplification which cannot be relied upon for design.

5.6 Governing Load Cases

As Hong Kong is in a region that is subjected to severe typhoons, wind loading dominates many aspects of the design. For the bridge deck, towers and piers the effects of the turbulent wind were found to be more onerous than the seismic effects. However, for the Structural Integrity Limit State the combined effects of the severe earthquake and the liquefaction that this could cause were critical in design of the piled foundations.

6. CONCLUSIONS

This paper shows the approach used to determine the seismic design of a large flexible bridge structure in Hong Kong. The performance requirements are listed and the process followed to determine the appropriate seismic hazard levels and the method of structural analysis. Given the high typhoon wind loading requirements in Hong Kong it is not surprising that the wind load dominated the design of the superstructure of the bridge. The design of the pile foundations however were affected by the extreme seismic loading case especially if liquefaction occurs in the loose fill under the action of this extreme event.

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