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Evaluation of Site Effects in Hong Kong

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ABSTRACT: This paper describes an investigation into the potential seismic site response effects for the ground conditions in Hong Kong. A site classification map for the Hong Kong region has been produced using current IBC site classes. Published geological maps and detailed ground investigation information have been used for the classification. One-dimensional site response analyses have been carried out to determine how various representative types of site profiles of Hong Kong will potentially respond to earthquake ground motion. The response of soil profiles was investigated using three input earthquake ground-motion levels corresponding to bedrock motion with 50%, 10% and 2% probability of being exceeded in 50 years. The results are presented in terms of spectral ratios for the different Site Classes. A simplified manual procedure for rapid assessment of site response effects has been developed by the University of Hong Kong to produce displacement-based design response spectra. There are extensive areas of reclamation in Hong Kong and the potential for liquefaction in these areas, and other areas underlain by non-cohesive, saturated soils, was considered sufficiently high to warrant investigation. The methodology and results of a liquefaction assessment are presented in the final part of this paper.

KEYWORDS**:** Seismic site effects, soil, amplification, earthquake, response spectra, Hong Kong

1 INTRODUCTION

The Hong Kong Special Administrative Region is located in an area of low to moderate seismicity. The current codes of practice for building design in Hong Kong do not require any seismic considerations. This paper describes an evaluation of seismic site response effects in Hong Kong. The evaluation of the seismic hazard levels on bedrock in Hong Kong is described in a companion paper, Pappin *et al.* (2008).

The potential for local ground conditions to significantly affect ground motion characteristics has been recognised for some time. Evidence that short-period and long-period ground motions are amplified by different amounts and that these amplification levels also vary with ground motion amplitude is incorporated into the NEHRP and IBC procedures (FEMA 1997, ICC 2006).

The subsoil conditions in Hong Kong have been classified across the entire region using the current IBC-2006 Site Classes A to E (ICC 2006). Published geological maps, at 1:20,000 scale, in conjunction with detailed ground investigation information, in the form of 1,200 borehole logs have been used as the basis for the categorisation.

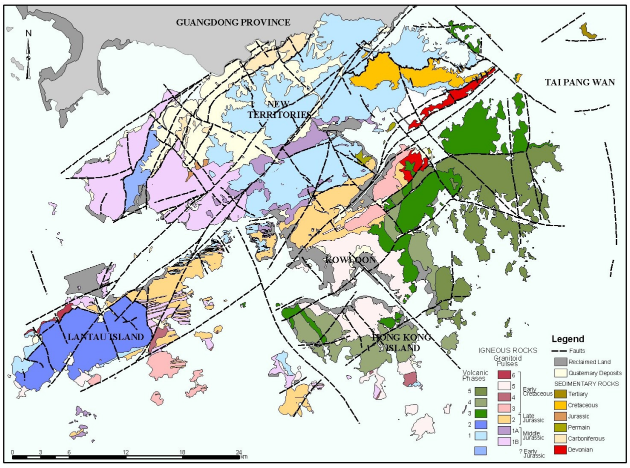

Figure 1. Simplified geological map of Hong Kong: modified after Sewell *et al.* (2000)

The potential for coastal reclamation and natural coastal deposits to liquefy under earthquake induced ground shaking has been observed in many past earthquakes worldwide, for example at the Port of Kobe, Japan in 1995. Due to the shortage of developable space in Hong Kong, there has been an extensive program of reclamation in place since the mid-19th century. The methodology and results of a liquefaction assessment undertaken for Hong Kong are presented in the final part of this paper.

2 CLASSIFICATION OF GROUND CONDITIONS IN HONG KONG

2.1 *Previous studies*

A number of Hong Kong site-specific site response analyses have previously been undertaken. The Geotechnical Engineering Office (GEO) of the Hong Kong Government published a pilot study for six representative sites around Hong Kong to explore the potential site amplification effects (GEO 1997). Some preliminary seismic site response analyses were carried out by Wong *et al.* (2000), who also determined dynamic properties for selected Hong Kong soils based upon both insitu field tests and laboratory tests. Chandler & Su (2000) present analyses of the seismic response of a number of reclamation sites in Hong Kong. Chandler *et al.* (2002) have investigated the soil amplification factors for soil sites in Hong Kong subject to ground motion from large magnitude, distant earthquakes, and have suggested that previous studies may have been unconservative in terms of estimating seismic amplification factors.

2.2 *Local geology*

The geology of Hong Kong is described by Sewell *et al.* (2000) and Fyfe *et al.* (2000) and is summarised in Figure 1. More than three-quarters of the land area of Hong Kong is underlain by igneous rocks predominantly volcanic tuffs and granites of Late Jurassic to Early Cretaceous age (140 to 120Ma). Older, Late Paleozoic age (420 to 240Ma) sedimentary rocks and younger Late Mesozoic to Tertiary age (140 to 2Ma) sedimentary rocks underlie the majority of the remaining land area. Superficial deposits comprising Quaternary age (less than 2Ma) alluvium and other unconsolidated deposits are also present throughout the territory. The Quaternary age deposits in Hong Kong comprise alluvium and colluvium materials deposited throughout the Middle and Upper Pleistocene and Holocene periods, covering a period of approximately 200,000 years. The sediments generally overlie bedrock, which has a weathered mantle of highly variable thickness. These weathered profiles are typically a few metres to several tens of metres but can exceed 200m locally (Fyfe *et al.*, 2000). Large reclamation areas are also present in Hong Kong (see Figure 1), with the history of reclamation extending back to before 1887. These deposits comprise sand fill, mixed rock and soil fill as well as construction and demolition debris and are typically quite variable both laterally and vertically.

2.3 *Ground investigation data*

A database of ground investigation data has been compiled using Government maintained archives at the GEO and in-house databases maintained by Arup Hong Kong. Over 1,200 borehole logs have been compiled into a GIS and analysed for the purposes of ground conditions categorisation. As would be expected the available ground investigation information coincides closely with the built up regions of Hong Kong. The borehole logs include, as a minimum, detailed soil profile descriptions and standard penetration test (SPT) results. In addition, shear-wave velocity profiles were collated at 27 locations from published and unpublished studies (Wong *et al.*, 1998, 2000; Kwong 1998; Lee *et al.*, 1998; Chan & Bell 2000; Tam 2002 and Arup 2002). The shear-wave velocity profiles were obtained using a range of methods including downhole, crosshole, seismic cone, and spectral analysis of surface waves (SASW). At some typical reclamation locations the shear-wave velocity profiles passed through 50m of soil and extended into bedrock (see Figure 2).

There is limited data on the shear-wave velocity of the bedrock over a significant depth. A shear-wave velocity profile passing through 2,000m/s at 50 to 100m below rockhead and 2,500m/s at 500m below rockhead has been defined based upon data by Tsang (2006) (see Figure 3). The Hong Kong region profile is compared with profiles for Western and Eastern North America by Boore & Joyner (1997). It is apparent that the Hong Kong region data falls between the bounds defined by Western and Eastern North America.

2.4 *Classification according to the IBC2006 site classification system*

The IBC2006 site class definitions were adopted in this study (refer Table 1). The IBC definition for sites less than 30m thick is also a function of site period because of the way the average shear wave velocity (V_s) value is calculated. The advantages of using the IBC site classification system for Hong Kong are that it is a well-established internationally recognised system that provides a means for direct comparison with studies outside Hong Kong.

The Site Class has been derived at 1,217 locations using SPT N data, shear wave velocities and/or undrained strength measurements. Figure 4 shows the locations of the ground investigation stations, in the vicinity of Hong Kong Island, where the site class values were evaluated. These results were then combined with the geological map to develop a site classification map (shown in Figure 4). The site class boundaries were found to broadly coincide with geological boundaries across the territory. In particular, the boundary between Quaternary and pre-Quaternary age materials tends to coincide with the boundary between Site Class C and Class D. The boundary between Site Class C and Class B is associated with the presence or not of a relatively thin layer of saprolite over bedrock, which was found to generally coincide with slope gradient. Site Class E is typically in areas where there is a considerable thickness of reclamation material and/or reclamation material overlying a considerable thickness of soils such as marine or alluvial deposit.

Table 1: IBC2006 Site Classification System

Site Class	Description	Average Properties in top 30m		
		Shear-wave velocity, V_{S30} (m/s)	SPT N (Blows/300mm)	Undrained shear strength, S_U (kPa)
A	Hard Rock	> 1500	Not applicable	Not applicable
B	Rock	760 - 1500	Not applicable	Not applicable
C	Very dense soil and soft rock	360 - 760	> 50	> 100
D	Stiff soil	180 – 360	15 – 50	50 – 100
E	Soft soil	< 180	< 15	< 50
		Plus any profile with more than 3m of soil having the following characteristics: Plasticity index, PI > 20% Moisture content, w ≥ 40% Undrained shear strength, S_U < 25kPa		
F	Any profile containing soils with one or more of the following characteristics: Soil vulnerable to potential collapse under seismic loading, e.g. liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. Peats and/or highly organic clays (H>8m of peat and/or highly organic clay) Very high plasticity clays (H>8m with PI>75%) Very thick soft/medium stiff clays (H>36m)			

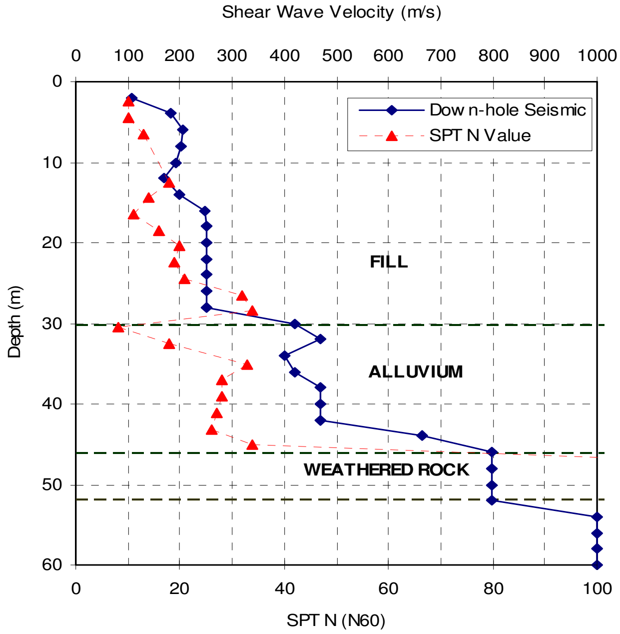

Figure 2. Typical shear wave velocity profile at reclamation site.

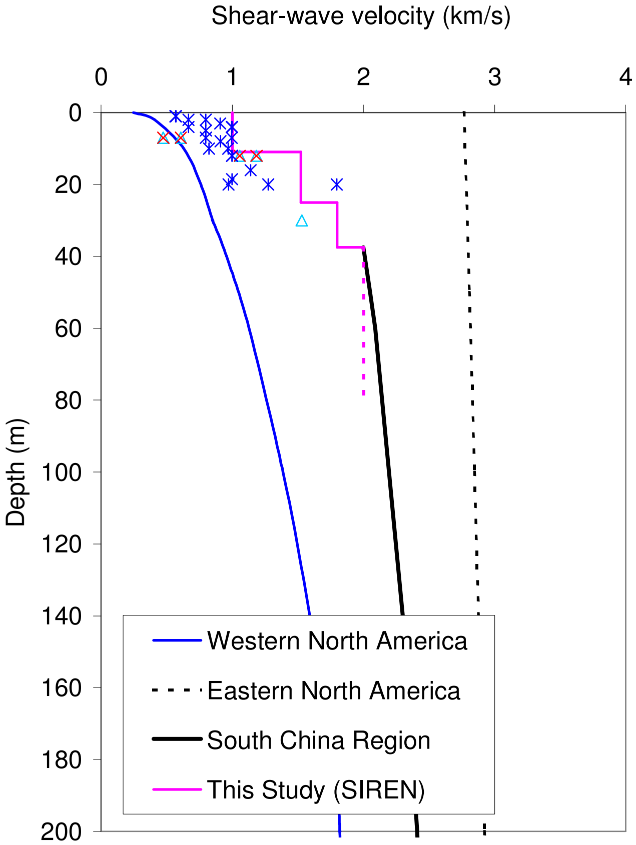

Figure 3. Bedrock shear wave velocity profiles (depth below bedrock).

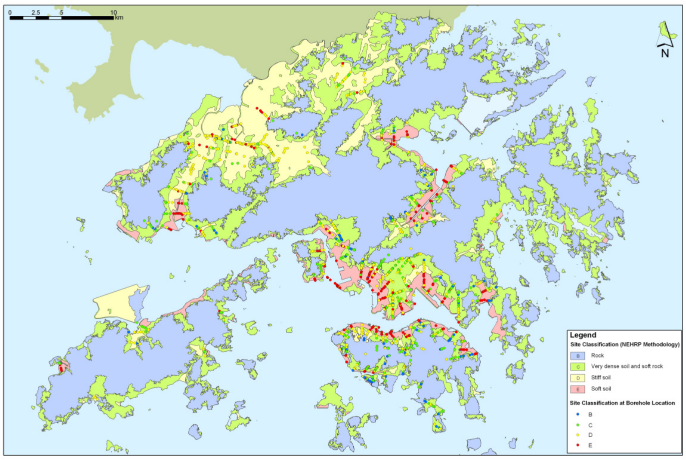

Figure 4. Site classification map for Hong Kong.

3 SITE AMPLIFICATION FACTORS

In order to investigate the seismic site response characteristics of the Hong Kong ground conditions, a series of one-dimensional site response analyses were undertaken for a wide range of soil profiles and input parameters. These results were then used to determine ground-motion amplitude- and period-dependent factors for each of the site classes. The factors presented herein have the advantage of being specific to Hong Kong in terms of the local soil conditions and the amplitude of the reference input motion. Due to its moderate level of seismicity, the input motions in Hong Kong are considerably lower than most of those used to derive the amplification factors for the IBC 2006.

3.1 1-Dimensional site response analysis methodology

The site response effects were analysed using the following steps:

- 41 representative soil profiles were compiled to represent the range of ground conditions encountered in Hong Kong (6 Site Class B, 7 Site Class C, 15 Site Class D and 13 Site Class E). Each profile is defined in terms of the soil and rock material types encountered and the variation in small strain shear modulus (G_0) versus depth. In each case the profile extends into bedrock.
- The shear modulus degradation curve, representing the non-linear behaviour of the soil was defined for each soil and rock material type, as was the soil density. The main soil types encountered were reclamation fill sand, marine silt/clay, colluvium sand and gravel deposits, alluvium sand and silt/clay deposits and in situ weathered rock or saprolite.
- Bedrock response spectra, representative of seismic hazard levels in Hong Kong were used to define appropriate earthquake strong-motion records for input as reference bedrock ground motions.
- One-dimensional site response analyses were carried out using the program *Oasys* SIREN.
- Spectral ratios were determined for each soil profile.
- Design surface response spectra were obtained by multiplying the bedrock design response spectra by amplitude dependent spectral ratios for a range of periods.

Oasys SIREN is the Arup in-house program for analysis of the response of a one-dimensional soil column subjected to an earthquake motion at its base. 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).

Stiffness degradation curves for site response analyses were defined for each material type. Limited test data from Hong Kong was used to confirm the appropriateness of well-established published stiffness degradation curves where possible. For reclamation fill (mainly sand) and alluvial sands, the curves of Seed & Idriss (1970) and Seed *et al.* (1986) were used. For marine and alluvial clays and silts, the curves of Vucetic & Dobry (1991) were used. For colluvium (mainly gravelly sands), the curves of Seed *et al.* (1986), for gravelly sands were used. For the in situ weathered rocks (saprolite), which underlie a large proportion of the region, a suite of cyclic triaxial tests was undertaken by the Hong Kong University of Science and Technology and a best-estimate curve drawn through the test data as shown in Figure 5.

Table 2. De-aggregated earthquake scenarios for Hong Kong

Probability of being Exceeded	Return Period (years)	Period (s)	M-d pair
50% in 50 years	72	0.2s	M = 5.8, d = 140km
		2.0s	M = 6.8, d = 426km
10% in 50 years	475	0.2s	M = 5.8, d = 60km
		2.0s	M = 7.3, d = 350km
2% in 50 years	2,475	0.2s	M = 6.3, d = 50km
		2.0s	M = 7.5, d = 300km

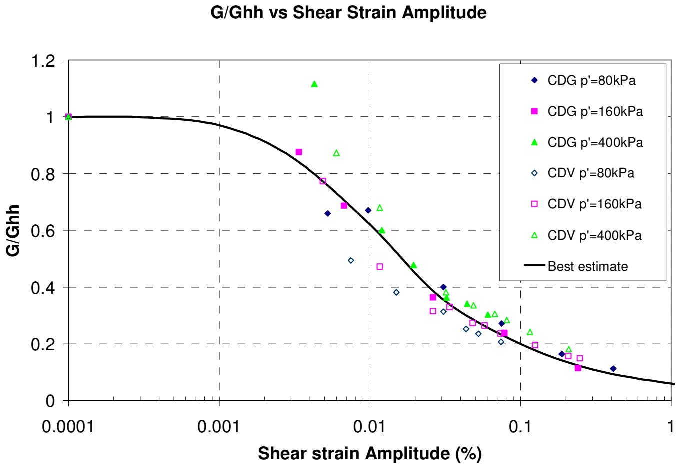

Figure 5. Stiffness degradation curve for saprolite soils in Hong Kong.

3.2 *Seismic hazard*

Seismic hazard levels for Hong Kong were defined in terms of uniform hazard response spectra (UHRS) for 50%, 10% and 2% probabilities of being exceeded in 50 years (Pappin *et al.*, 2008). The de-aggregation process recommended by McGuire (1995) was then applied to determine the earthquake magnitude and distance pairs contributing most significantly to different portions of the UHRS curves. Two scenario earthquakes for each UHRS were defined to represent the short and long period portions at approximately 0.2 seconds and 2.0 seconds. The magnitude distance pairs defined for Hong Kong are summarised in Table 2.

3.3 *Input ground motions*

In the absence of appropriate measured strong-motion records in the South China region, time histories from stable continental regions and elsewhere have been modified by adjusting them to a target spectrum in the frequency domain. The target design spectra were represented by the scenarios shown in Table 2. The input time histories were selected in order to minimise the degree of adjustment needed, by searching for magnitude and distance pairs similar to the de-aggregated combinations and minimising the scaling required to match the amplitude of the target spectra.

3.4 *Results of site response analysis*

For each input time history and each soil class, the mean spectral ratio was assessed at structural periods of 0.01, 0.1, 0.2, 0.5, 1.0, 2.0 and 5.0 seconds by calculating a running average spectral ratio, which was found to give a better representation of the site response effects and reduce the influence of minor perturbations of the spectral ratio with change in period. Figure 6 shows the calculated spectral ratios for Site Class D sites when subjected to the short period 2% in 50-year ground motion for example.

Figure 7(a) shows the average amplifications of the short period 0.2 second ground motions. The predicted amplifications are between about 1 and 3 times, with greater amplifications for Site Class C than for Site Classes D and E. As the input ground motion increases, the amplification of the soft soil sites decreases, until eventually a reduction is shown. This result is consistent with the non-linear response expected for soft soil profiles. Figure 7(b) shows that, on average, the long period 2 second motion is not amplified for Site Classes B and C, irrespective of the amplitude of the input motion. Moderate amplification is found for Site Classes D and E of 1.4 to 2 times. For Site Class E, the amplification continues to increase slightly as the amplitude of motion increases, contrary to expectations. The design ground motions for Hong Kong comprise very low energy long period motions, which may explain these results. At longer structural periods of 5.0 seconds, little or no amplification was found for all site classes.

The relationships developed for spectral ratio versus bedrock spectral acceleration for each site class have been used to generate surface spectra for each site class. The resulting ground surface velocity uniform hazard response spectra (UHRS) are shown in Figures 8 and 9. The median (solid lines) are shown for ground motion levels with 10% and 2% probability of being exceeded. These results have been previously published in Pappin *et al.* (2004).

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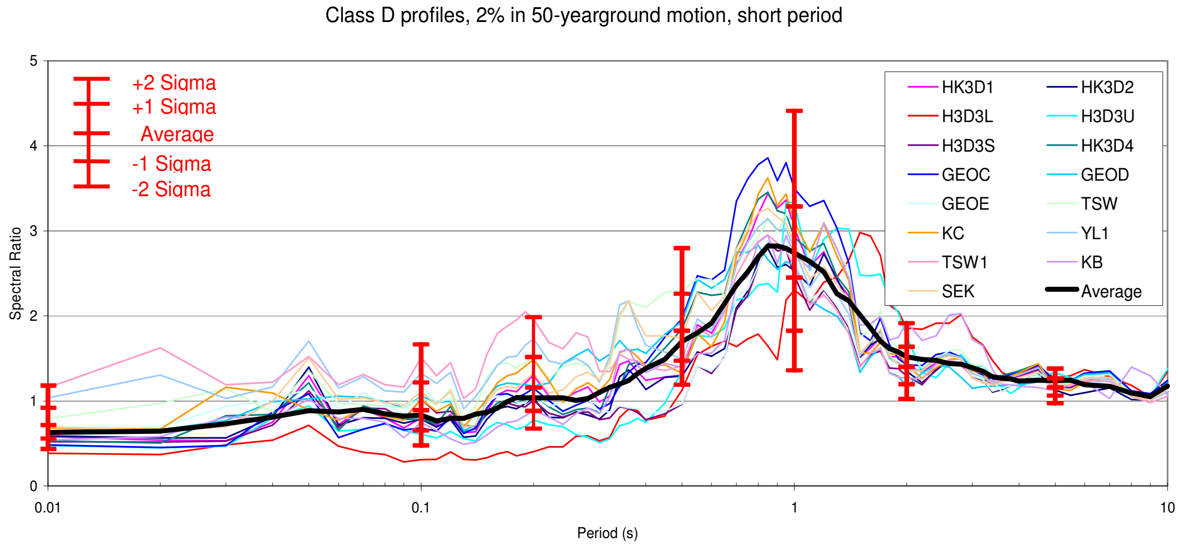
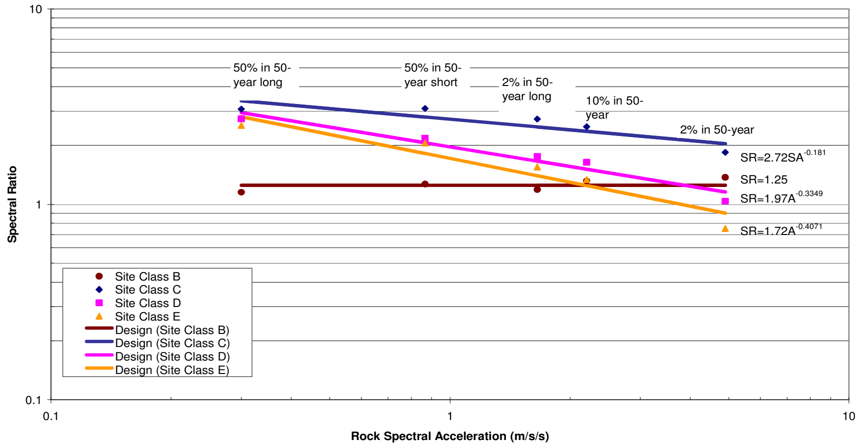
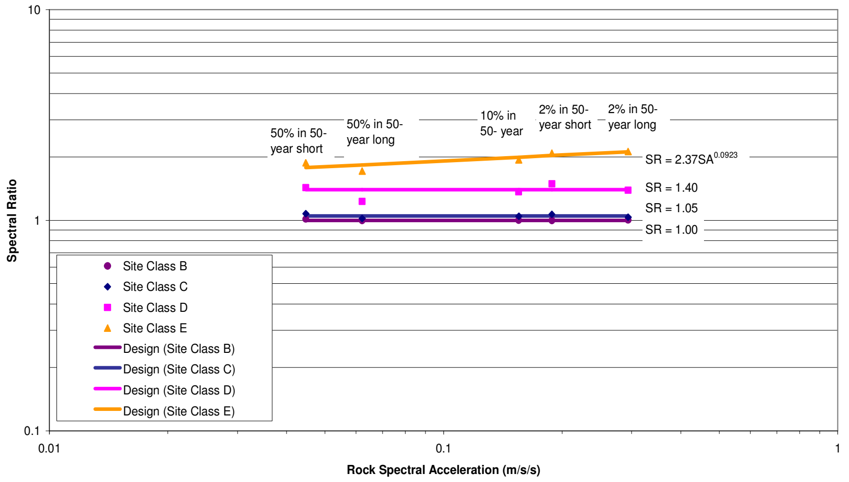


Figure 6. Example spectral ratios for the 2% in 50-year ground motion at sites in Class D.



(a) Structural period of 0.2 seconds



(b) Structural period of 2 seconds

Figure 7: Mean spectral ratio values at (a) 0.2 second and (b) 2.0 second structural periods.

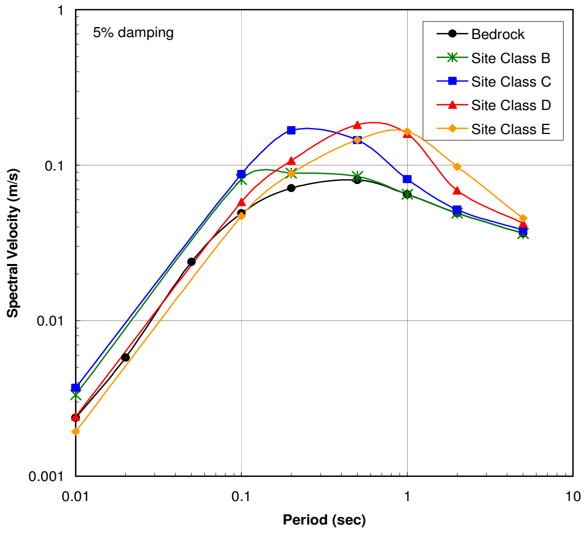


Figure 8. Horizontal velocity UHRS for 10% in 50-year ground motion

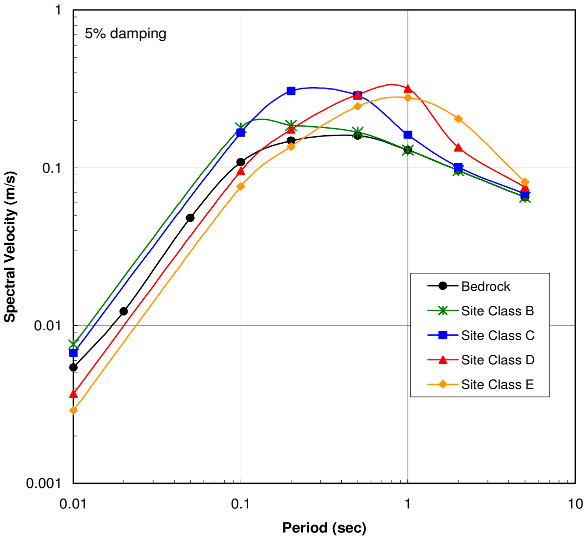


Figure 9. Horizontal velocity UHRS for 2% in 50-year ground motion.

3.5 *Assessment of variability and sensitivity of the results*

Figure 6 shows a range of amplification results for the sites comprising Site Class D. The standard deviation of this variability was determined for all soil classes at all structural periods. At each structural period the standard deviations were observed to be a function of the mean spectral amplification and to be virtually independent of the site class.

Sensitivity analyses were undertaken to investigate the effect of the uncertainty in soil stiffness parameters determined for the different soil materials. These sensitivity analyses considered the potential variability of the parameters by taking mean, lower bound and upper bound stiffnesses. The results indicated changes in spectral ratio values of 10 to 50% in the short period range and 10 to 25% in the long period range. It was considered that the variability determined as described above is sufficient to encompass these results

4 RAPID ASSESSMENT PROCEDURE

The results above are for general understanding of the effects of local soil condition on the seismic risk. The actual seismic risk level at an individual site may be underestimated and allowance should be made for the variability in the site amplification. It is suggested that the mean plus one standard deviation amplification should be used. Alternatively a site specific one dimensional dynamic analysis such as SHAKE or SIREN could be used. Recently a simplified manual procedure has been developed in the University of Hong Kong (Tsang *et al.*, 2006a) to provide an alternative convenient tool for rapid site response analysis for individual site and can be expedited by the use of a spreadsheet program.

The key element of this technique is the use of the spectral ratio (*SR*) at the site natural period T_g (considering the period-shift effect). It is defined as the ratio between the maximum spectral velocity (or displacement) of soil spectrum ($RSV_{\max}$ or $RSD_{\max}$ at period T_g) and the spectral velocity (or displacement) of bedrock spectrum at period T_g (RSV_{Tg} or RSD_{Tg}), and has been proposed to be composed of two components: the nonlinear peak velocity (or displacement) ratio (*PVR* or *PDR*) and a resonance factor $f(\alpha)$:

$$SR = \frac{RSV_{\max}}{RSV_{Tg}} = PVR \cdot f(\alpha) \qquad (1)$$

where $$PVR = \frac{2\alpha}{1+\alpha}\sqrt{\frac{\beta}{1-R^4\beta^4}} \qquad (2)$$

which is defined as the ratio between the peak ground velocity on the surface of the soil sediment and that on the bedrock surface. Equations (1) and (2) were developed theoretically based on basic wave theories and the concept of multiple reflections of seismic waves within the soil sediments. The accuracy of these two equations has been demonstrated by a comprehensive comparison with results obtained from nonlinear shear wave analyses and data recorded during the 1994 Northridge earthquake (Tsang *et al.*, 2006a).

Also, $f(\alpha) = \alpha^{0.3} \le 2.3$ (3)

$$\alpha = \frac{\rho_R V_R}{\rho_S V_S} \text{ is the impedance ratio,} \tag{4}$$

in which, ρ and V are the weighted-average of the density and the shear-wave velocity of the respective layers, and the subscripts R and S represent the rock and soil layers, respectively. It is noted that, theoretically, the averages of the seismic velocity (V) and density (ρ) should be determined independently, instead of averaging the seismic impedance.

R is the reflection coefficient, which describes the amplitude ratio of the upwardly propagating reflected wave and the downwardly propagating incident wave within the soil layer, and is expressed by:

$$R = \frac{1-\alpha}{1+\alpha} \tag{5}$$

The parameter β is related to the energy dissipation within the soil layer, and is defined as:

$$\beta = \exp(-\pi\zeta) \tag{6}$$

where ζ is the soil damping ratio and can be estimated by (Tsang *et al.*, 2006b):

$$\zeta = 12.5 + 6.5\log(R_\gamma \lambda \psi) - 0.13PI \tag{7}$$

$$\text{where } \psi = \frac{RSV_{Tg}}{V_S} = \frac{RSD_{Tg}}{H}\frac{\pi}{2} \tag{8}$$

R_γ is the ratio of the effective shear strain to maximum shear strain, which has been empirically found to vary between about 0.5 to 0.7 (0.6 has been used in this study). *PI* is the plasticity index of soil material (in %). *H* is the thickness of the soil layer.

The reduction factor λ is needed to account for the bedrock rigidity effect:

$$\lambda = \frac{\alpha}{1+\alpha}\sqrt{\frac{1-\beta^4}{1-R^4\beta^4}} \tag{9}$$

Equation (7) may be bounded by a "practical" minimum damping ratio ζ_{pi} and an upper bound damping ratio ζ_{ub}:

$$\zeta_{pi}(\%) = 2.5 + 0.03 \cdot PI(\%) \le 6.8 \tag{10}$$

$$\zeta_{ub}(\%) = 17.5 - 0.07 \cdot PI(\%) \ge \zeta_{pi} \tag{11}$$

Finally, the period shift ratio can be estimated by (Tsang *et al.*, 2006b):

$$\frac{T_g}{T_i} = 1 + R_\gamma \lambda \psi \mu \tag{12}$$

The parameter μ, termed the "Plasticity Factor", is used to allow for different *PI*, as shown in Table 3.

Table 3. The Plasticity Factor μ [refer Equation (12)] for different plasticity indices (*PI*)

Plasticity Index *PI* (%)	0	15	30	50
Plasticity Factor μ	1.6	0.9	0.4	0.2

4.1 Displacement response spectrum (RSD) modelling

The simple procedure for constructing site-specific *RSD* is described as follows:

1. Obtain the basic parameters from normal site investigation: initial shear-wave velocity V_{Si} (or *SPT "N"* value) and *PI*, thickness *H* of the soil layer; bedrock shear-wave velocity V_R. The initial site natural period T_i can then be computed by Equation (13). RSV_{Ti} or RSD_{Ti} can be obtained from rock response spectrum (e.g. Pappin *et al.*, 2008).

$$T_i = \frac{4H}{V_{Si}} \tag{13}$$

2. Calculate the soil damping ratio, by Equation (7), with λ = 1; $RSV_{Tg} = RSV_{Ti}$ (or $RSD_{Tg} = RSD_{Ti}$), $V_S = V_{Si}$; and then the damping factor β, by Equation (6), and the reduction factor λ, by Equation (9). The shifted site natural period T_g can be computed by Equation (12).
3. Calculate the degraded soil V_S , by Equation (14), with the consideration of the following relationship:

$$\frac{V_S}{V_{Si}} = \frac{T_i}{T_g} \tag{14}$$

   RSV_{Tg} (or RSD_{Tg}) can then be obtained from rock response spectrum. (It is noted that if RSD_{Tg} is used in the calculation, it is not required to compute the degraded soil V_S .) Then, the actual shifted site natural period T_g, using the degraded soil V_S , can be computed again.
4. The impedance ratio α [Equation (4)], the reflection coefficient R [Equation (5)] and the soil damping ratio [Equation (7)], together with the

damping factor β [Equation (6)] can then be calculated.

5. The $RSV_{\max}$ or $RSD_{\max}$ can be computed by Equations (1) – (3), and the two parameters can also be linked by Equation (15):

$$\frac{RSD_{\max}}{RSV_{\max}} = \frac{T_g}{2\pi} \quad (15)$$

6. The idealised bi-linear site-specific *RSD* can be constructed for any given soil site, as demonstrated in Figure 10.

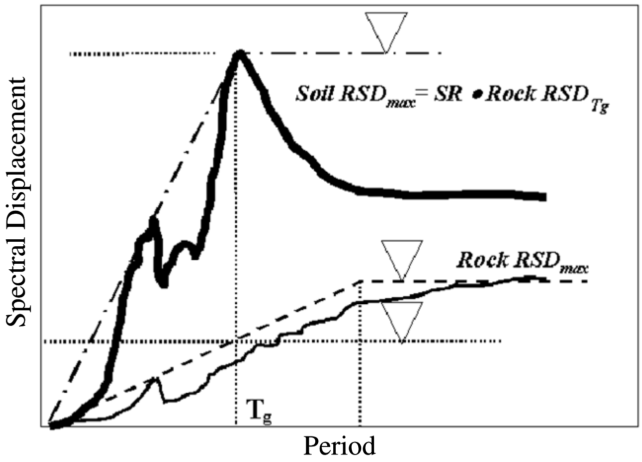


Figure 10. Idealised bi-linear site-specific displacement response spectrum (RSD) model.

Figures 11 and 12 compare the *RSD* computed from SIREN with the proposed idealised bi-linear model, for two reclamation sites in Central (CL) – Site Class D, and Tseung Kwan O (TKO) – Site Class E, respectively. [Details: V_{Si} = 317 m/s & 214 m/s; *PI* = 21% & 25%; V_R = 2000 m/s for both sites]. The input rock motion is based on two far-field earthquake scenario events for 475 (M = 7.3, d = 350km) and 2475 (M = 7.5, d = 300km) years of return periods (Pappin *et al*., 2008). It is shown that the idealised bi-linear model can effectively capture the resonance peak displacement demand and the corresponding site natural period. The proposed model, while being conservative, can be used as an efficient tool for estimating site-specific site response effects for longer period ground motions.

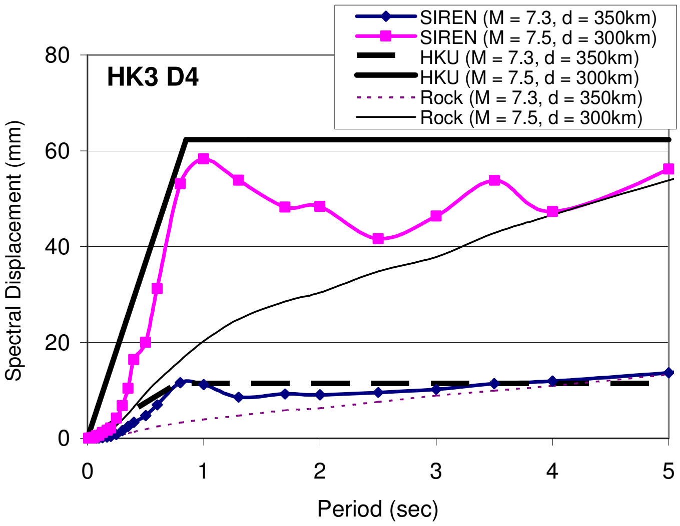


Figure 11. Comparison between the idealised bi-linear *RSD* model with the computed result from SIREN(Central).

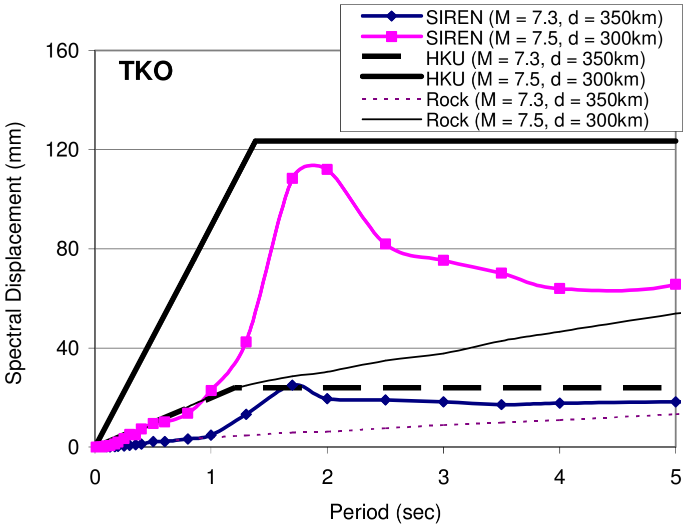


Figure 12. Comparison between the idealised bi-linear *RSD* model with the computed result from SIREN(Tseung Kwan O).

5 LIQUEFACTION ASSESSMENT

Liquefaction susceptibility can be determined from historical precedence, or empirical criteria related to the type and age of various soil deposits. There is no historic evidence of liquefaction in Hong Kong due to its relatively low historical seismicity. This does not however preclude the occurrence of liquefaction in future earthquakes, particularly since there has been no strong shaking since much of the development and reclamation were carried out. Therefore geological and geotechnical information for Hong Kong have been used to identify which soil layers should be studied further.

The most prevalent soil deposits found within the top 30m in Hong Kong are listed in Table 4, along with their considered susceptibility to liquefaction. The ratings presented by Iwasaki *et al.* (1982) regarding the susceptibility of geomorphological units to liquefaction were referred to, in combination with knowledge of Hong Kong conditions.

It follows that only reclamation fill, alluvium deposits and marine deposits should be considered in the liquefaction assessment. Reclaimed fills in Hong Kong are very variable depending on their age and their parent material. In some regions the fill contains construction debris and decomposed rock whereas in other areas it is composed of clean sand excavated from marine borrow areas and deposited in a loose state. It is the clean sand fill that is considered as potentially liquefiable. Both alluvium and marine deposits can vary between gravels sands, silts and clays. It is assumed that the clays will not liquefy. The liquefaction of silts is an area of ongoing research but there is sufficient evidence to show it can occur and therefore sites with alluvial silts, sands and gravelly sands were considered in this assessment.

Twenty two soil profiles have been used to represent a range of typical sites where reclamation fill, alluvium and marine deposits are present, for the purposes of a more detailed liquefaction assessment. All of these were classified as either Site Class D or Site Class E. There are significant differences between the sites, all of which are coastal, relating to the thickness of the material type, thickness, density and fines content, and the depth to bedrock. The selection represents the natural variability of the Hong Kong coastal deposits and therefore the results reflect the range of expected behaviour.

5.1 Probabilistic method of assessing liquefaction potential

The probabilistic method by Seed *et al.* (2003) for the evaluation of liquefaction potential has been used. Their relationship is summarised in the following equation:

$$P_L = \Phi\left[-\frac{\begin{pmatrix} N_{1,60}\cdot(1+0.004\cdot FC)-13.32\cdot\ln(CSR) \\ -29.53\cdot\ln(M_w)-3.70\cdot\ln(\sigma_v') \\ +0.05\cdot FC+33.73 \end{pmatrix}}{2.7}\right] \quad (16)$$

where P_L = the probability of liquefaction;
Φ = the standard cumulative normal distribution;
$N_{1,60}$ = standard penetration test (SPT) N blow count corrected for overburden effects and for energy;

$$N_1 = \left(\frac{100}{\sigma_v'}\right)^{0.5} N \quad (17)$$

$$N_{1,60} = N_1 \cdot C_R \cdot C_S \cdot C_B \cdot C_E \quad (18)$$

C_R = correction for rod lengths L shorter than 10m (0.75 for $L < 3$m, 0.8 for $L = 3 – 4$m, 0.85 for $L = 4 – 6$m, 0.95 for $L = 6 – 10$m, 1.0 for $L > 10$m);
C_S = correction for non-standard SPT samplers;
C_B = correction for borehole diameters > 115mm;
C_E = correction for energy efficiency of hammer;
(C_S , C_B , C_E are equal to unity in local practices.)
FC = fines content (percent finer than 0.074mm);
M_w = moment magnitude of the earthquake for which the liquefaction probability is being assessed;
σ_v' = effective vertical stress (kPa);
CSR = 'equivalent uniform cyclic shear stress ratio'. This has been evaluated by means of site-specific seismic site response analyses using SIREN, described previously. SIREN computes the peak shear stress (τ_{max}) at each node, and the 'equivalent uniform' CSR is assumed to be equal to 65% of the peak, where

$$CSR_{eq} = 0.65 \cdot \frac{\tau_{\max(SIREN)}}{\sigma_v'} \quad (19)$$

The SPT N values, fines contents and unit weights were taken from the representative boreholes. The natural variability of the in situ density measurements was represented through the use of the 22 different boreholes, therefore analysis of a single profile at each site was considered sufficient.

The earthquake scenarios representative of the 50%, 10% and 2% probabilities of being exceeded in 50 years that were used for the 1-dimensional site response analyses (see Table 2), have also been used for the liquefaction assessment. Since both the amplitude of shaking and the number of cycles of shaking (or the duration of shaking), contribute to the development of liquefaction, this approach, which uses site-specific site response analysis with carefully selected earthquake scenarios is believed to provide a good estimate of the probability of liquefaction.

Table 4: Liquefaction susceptible soil deposits in Hong Kong

Deposit	Variability	Distribution	Locations	Susceptibility Rating
Reclamation Fill (Sand, Mixed Rock/Soil, Construction Debris)	High	Moderate	Developed coastal zones	Likely
Marine Deposits (Silt/Clay, Sand)	High	Limited	Reclaimed coastal zones	Possible
Alluvium Deposits (Silt/Clay, Sand, Gravel)	High	Widespread	Coastal and valley bottoms	Possible
Colluvium Deposits (Clayey Gravel, Sand, Gravely Sand)	Very high	Moderate	Hillsides and valley bottoms	Unlikely
Decomposed Rock (Clayey Sand, Silty Clay)	Moderate *	Widespread	Everywhere	Unlikely

*Variability dependent on parent rock type

Typical results of the probabilistic liquefaction assessment are shown in Figure 13 in terms of the probability of liquefaction versus depth for each borehole. The liquefaction risk for the 50% or 10% probability of being exceeded earthquake hazard levels was negligible and only the 2% probability of being exceeded in 50 years earthquake scenarios were found to result in a significant probability of liquefaction. In particular, the result for the long-period scenario ground motion dominated the probability of liquefaction, due to the larger magnitude associated with this event and greater shear stresses induced in the soil. The results for the long-period scenario event show that about 60% of the sites have a probability of liquefaction of 60% or greater at depths down to 15m. The results reflect the natural variability of soils and the scatter in the SPT N measurements, in that even where a soil type is considered highly susceptible to liquefaction, the actual in situ susceptibility can vary greatly. Figure 13 shows the great variability of the results for sites in Class E for example. Overall the average probability of liquefaction in the upper parts of sites in Class E was found to be approximately 35%. The liquefaction potential is higher in the upper portions of the profiles, which nearly always coincides with reclamation fill material.

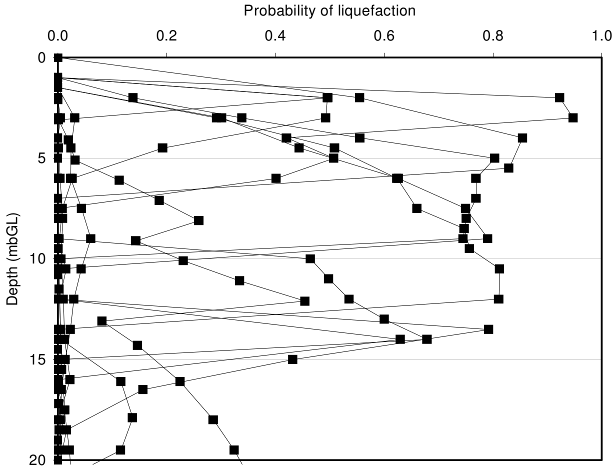


Figure 13. Probability of liquefaction for the long period 2% in 50 year ground motion for sites classified as Site Class E.

6 CONCLUSIONS

Site classification, using the system defined in the IBC Provisions, has been undertaken for the subsoil conditions in Hong Kong. The classification system defines six site classes (Site Classes A to F) based upon the soil / rock type, thickness and average geotechnical properties in the upper 30m of the ground profile. Over 1,200 borehole logs have been examined and analysed to determine the site classification at individual locations. A site classification map has been produced, which shows the distribution of the different site classes across Hong Kong.

Site response analyses have been undertaken for earthquake ground motion levels with 50%, 10% and 2% probability of being exceeded in 50 years. Spectral ratios have been presented for the different site classes with respect to the level of the input bedrock motion such that site response effects can be determined for a range of scenario input ground motion levels within appropriate limits. The site response effects are found to be dependent on both structural period and input amplitude with maximum mean spectral ratios up to about 3. The variability associated with the site response analyses has also been examined by determining the standard deviation from the mean spectral ratio values. The variability is also found to be a function of the level of spectral amplification for each structural period.

A simplified manual procedure has been presented to provide an alternative convenient tool for rapid site response analysis for individual site and can be expedited by the use of a spreadsheet program.

The potential for liquefaction under seismic ground motion loading has also been investigated. It has been shown that only reclamation sites, where loose, saturated sands are present, have a significant probability of liquefying. For the 2% in 50-year ground motion, there is a risk of liquefaction in the most susceptible sites.

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