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Seismic Hazard Modelling for Hong Kong

* Nelson Lam [1)] and Hing-Ho Tsang [2)]

[1)] Infrastructure Engineering, Melbourne School of Engineering,
The University of Melbourne, Parkville, VIC 3010, Australia
[1)] ntkl@unimelb.edu.au

[2)] Department of Civil & Construction Engineering, Swinburne University of Technology,
Melbourne, VIC 3122, Australia (formerly with The University of Hong Kong)
[1)] htsang@swin.edu.au

ABSTRACT

This paper is concerned with the development of a seismic action model for structural design purposes in Hong Kong. The development of the seismic hazard model for Hong Kong is first reviewed. The choice of suitable performance criteria of the structure and the return period of the design seismic actions is then considered. The determination of the design response spectrum and the associated scaling factors to take into account the ultimate performance behaviour of the structure and its importance classification is also discussed. Site specific seismic action model which takes into account subsoil conditions is a topic of great significance in the Hong Kong context given the extent of reclaimed land found on unconsolidated marine deposits.

1. INTRODUCTION

Seismic design provisions for bridges in Hong Kong have been around for a long time. Meanwhile, Hong Kong is without an official regulatory document which defines minimum design requirements of building structures in relation to the consideration of seismic hazard for privately funded construction projects. Although Hong Kong is located some 600 km away from the nearest tectonic plate boundary the level of seismicity is actually higher than many parts of Australia where seismic design provisions have been mandated for over two decades. Research into the seismic hazard of Hong Kong has commenced for more than two decades. Probabilistic seismic

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[1)] Professor
[2)] Senior Lecturer

hazard analysis conducted by *ARUP and The University of Hong Kong* in recent years has culminated in the development of *Uniform Hazard Spectrum* models that can be codified for the structural design of buildings and other types of structures.

Capitalising on this achievement is the key motivation behind the writing of this paper which is aimed at providing recommendations on key decisions that need to be made when drafting the first edition of the earthquake loading standard for Hong Kong. This paper also contains recommendations for improvements over conventional codification practices in regions of low to moderate seismicity. Justifications for these recommendations were evolved from research that have continued for over two decades.

A review of the literature in relation to the development of the seismic hazard model for the region is first presented. Decision on the return period of the design seismic actions and the corresponding design peak ground acceleration value for buildings of different importance classes is then discussed.

The rest of this paper is divided into two parts:

- Elastic response spectrum models on rock and soil sites.
- Structural performance criteria and parameters for design seismic actions

Eurocode 8 (CEN 2004) will be used to guide the drafting of the future earthquake loading standard for Hong Kong. The adoption of *Eurocode 8* is justified in view of the fact that other parts of the *Eurocode* have already been adopted in Hong Kong for the design of concrete, steel and composite structures for certain types of construction. Furthermore, *Eurocode* 8 has already been implemented in Singapore (BC3, 2003; NA to SS EN199801, 2013) and will soon be implemented in Malaysia.

2. Response spectrum models on rock and Soil

Research into the seismic hazard affecting Hong Kong dates back to the early 1990's (e.g. Pun and Ambraseys 1992; Lee 1998). In those early investigations the ground motion predictive expressions employed in the prediction of hazard were mainly based on relationships developed from empirical database of records collected in the high seismicity region of California. Few evidences were available to support the argument that relationships employed in the modelling were representative of local conditions. Ground motion properties were represented simply by the peak ground acceleration of the earthquake. More recent investigations including that by Free (2004) took into account distinct features of ground motion attenuation properties in the more stable regions of low and moderate seismicity such as *Central and Eastern North America* (CENA) and many parts of China including *South China*.

A technique known as stochastic simulations of the seismological model which was originally developed in CENA for modelling earthquake ground motions enables reliable predictions to be made based on utilizing regional information that are related

to the wave generation, and transmission, properties of the earth crust (refer Lam 2000 for a review of the methodology for engineering applications). Central to the modelling methodology is the generation of artificial accelerograms on rock sites by computer (e.g. program GENQKE of Lam 1999) in order that the issue of lack of locally recorded strong motion data can be circumvented. This technique was adapted for modelling ground motions in South China which includes areas surrounding Hong Kong in particular (Lam 2002; Chandler and Lam 2002; Tsang 2006; Chandler 2006). Similar modelling techniques were adapted for developing attenuation relationships in other parts of China (Tsang 2010) and many other parts of the world including Australia, Iran, Singapore, Malaysia and India (e.g. Lam 2003, 2006 and 2009; Yaghmaei-Sabegh and Lam 2010; Chandler and Lam 2004). Importantly, intensity attenuation relationships developed from site reconnaissance data collected from historical earthquake events in these regions have been reconciled with predictions by the respective seismological model.

The anelastic attenuation model of Mak (2004) has been specifically developed to model conditions surrounding Hong Kong based on analyses of data from seismological monitoring conducted locally. Artificial accelerograms generated in accordance with this locally developed seismological model using program GENQKE enabled ground motion predictive relationships to be developed as if there were an abundance of locally recorded accelerograms (Pappin 2008). This ground motion predictive relationship has been described in the literature (Pappin 2015) as the "*ARUP – HKU* attenuation relationship (2006)" acknowledging joint efforts between *ARUP International* and *The University of Hong Kong* in collaboration with *The University of Melbourne* where program GENQKE was written.

Development of *Uniform Hazard Spectrum* (UHS) models for Hong Kong is currently dominated by the work of two research groups. The first research group is *The University of Hong Kong* and its *Uniform Hazard Spectrum* (*HKU UHS*) model was first published in Tsang (2006), and was also presented in Tsang (2009) and Tsang and Lam (2010) based on the location of *Tsim Sha Tsui*. The *Direct Amplitude-Based* (DAB) methodology of Tsang and Chandler (2006) was employed to model seismic activities in the development of the *HKU UHS model*.

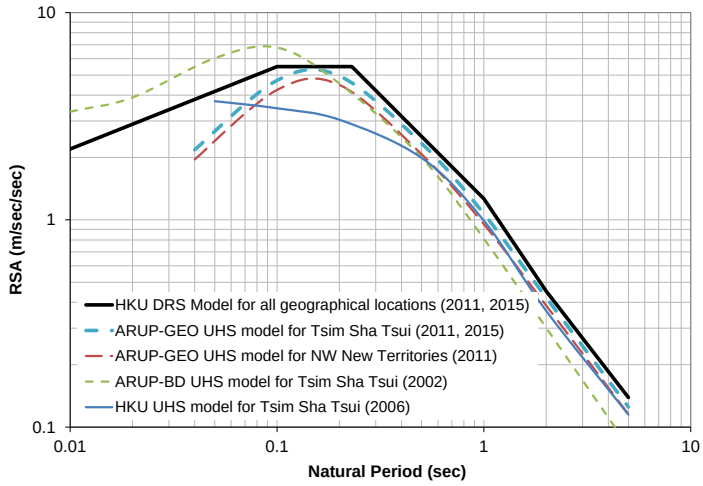
The second research group is ARUP which was consultant to the *Buildings Department* (BD) in 2002, and the *Geotechnical Engineering Office* (GEO) in 2011. The recommended UHS contained in the report submitted by ARUP (2002) is denoted herein as the *ARUP-BD UHS* model whereas that submitted by ARUP (2011) the *ARUP-GEO UHS* model. In the study undertaken in 2011 ARUP and GEO was also in collaboration with the *Guangdong Seismological Bureau* to make recommendations over seismic hazards for the *Northwest New Territories* of Hong Kong. Seismic hazard models for other parts of Hong Kong have also been developed during this time and presented in the form of contour maps in ARUP (2011), subsequently published in Pappin (2015). The conventional *probabilistic seismic hazard analysis* (PSHA) methodology, which was pioneered by Cornell (1968) and McGuire (1995), was adopted in studies by ARUP.

Good consistencies between the independently developed *HKU UHS* and the *ARUP-GEO UHS* models in the medium, and high, natural period range can be seen in Figs. 1(a) – 1(c). Both studies made use of the *ARUP – HKU* (2006) attenuation relationship which was underpinned by stochastic simulations of the seismological model by program GENQKE. In the 2011 study by ARUP, the *ARUP – HKU* (2006) attenuation relationship was assigned a 50% weighting factor whereas much lower weighting factors were assigned to other relationships developed elsewhere in North America and China. Different assumptions associated with seismic source modelling by the two groups have resulted in minor discrepancies between their recommendations.

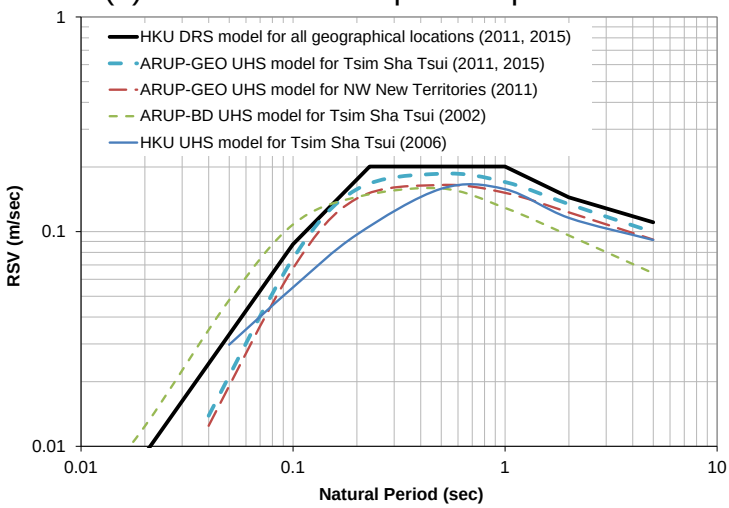
As the hazard levels in the *ARUP-GEO UHS* model vary by 20 – 30% across Hong Kong (given the different distances from the potential earthquake sources), a *Design Response Spectrum* (*HKU DRS*) model was derived to take into account spatial variations of seismic hazard within Hong Kong (Su 2011; 2015). The *HKU DRS* model is essentially an envelope which has incorporated findings from both the *HKU* and the *ARUP-GEO UHS* models. The *HKU DRS* model as shown by the dark bold line in Figures 1(a) – 1(c) is recommended for future codification for Hong Kong (refer Appendix A for details). Given a maximum response spectral acceleration (i.e. highest ordinate of the spectrum in Figure 1a) of 5.5 m/sec^2, or 0.56g, at a return period of 2500 years the corresponding peak ground acceleration value is 0.22g approximately.

Response spectra for soil sites are based on a site factor model which takes into account effects pertaining to resonance behaviour on flexible soil deposits. This is a major topic which warrants a separate presentation to cover. Figs. 2(a) & 2(b) present schematic representation of the construction of the response spectrum, SD(T), in the displacement format for stiff and flexible soil sediments respectively.

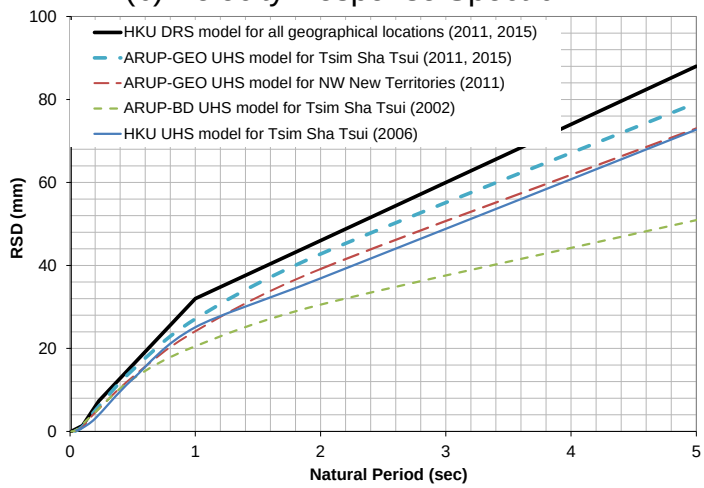
These figures are excerpt from a paper presented recently in an earthquake workshop held in Kuala Lumpur (Lam 2015) and is based on the theoretical model published in Tsang (2006a; 2006b) and is in accordance with the response spectral shape as proposed in Lam (2001). The intention of introducing this new site response model is to circumvent the need of design engineers to conduct dynamic analysis of the subsoil model of the site which can be a very labour intensive exercise in a normal design office setting. The soil response spectrum in each figure is shown alongside the corresponding response spectrum on rock, $SD_R(T)$. The value of parameter S which characterises amplification on the velocity controlled region of the response spectrum is recommended to be 1.5 for stiff soil sites (Tsang 2017a) and 3.6 for flexible soil sites (Tsang 2017b). The initial site natural period (T_s) at low amplitude of oscillation is either measured by geophones or by analysis of information reported in a standard borelog.



(a) Acceleration Response Spectrum



(b) Velocity Response Spectrum



(c) Displacement Response Spectrum

Fig. 1: Uniform Hazard Spectrum models for Hong Kong rock sites

Literature references for every item on the legend in Figs. 1(a) – 1(c) are listed as follows:

HKU DRS model for all geographical locations (2011, 2015)
– Su (2011 & 2015)
ARUP–GEO UHS model for *Tsim Sha Tsui* (2011, 2015)
– Report by ARUP (2011) and Pappin (2015)
ARUP–GEO UHS model for NW New Territories (2011)
– Report by ARUP (2011).
ARUP–BD UHS model for Tsim Sha Tsui (2002)
– Report by ARUP (2002).
HKU UHS model for Tsim Sha Tsui (2006)
– Tsang (2006), Tsang & Chandler (2006), Tsang (2009) & Tsang and Lam (2010)

Conversion from the response spectral displacement (RSD) presented in Figs. 2(a) & 2(b) to response spectral velocity (RSV) and response spectral acceleration (RSA) can be facilitated using Eqs. 1(a) & 1(b) respectively.

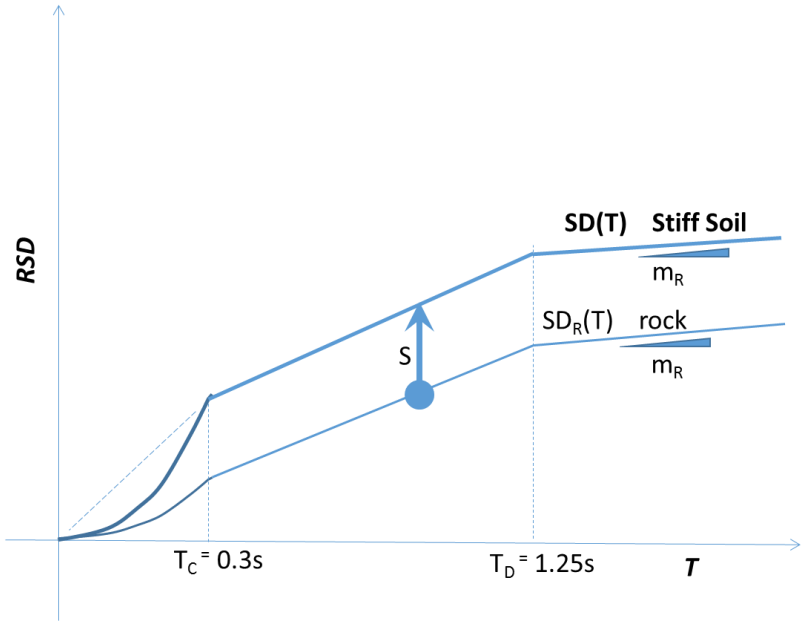
$$RSV = RSD \times \frac{2\pi}{T} \tag{1a}$$

$$RSA = RSD \times \left(\frac{2\pi}{T}\right)^2 \tag{1b}$$

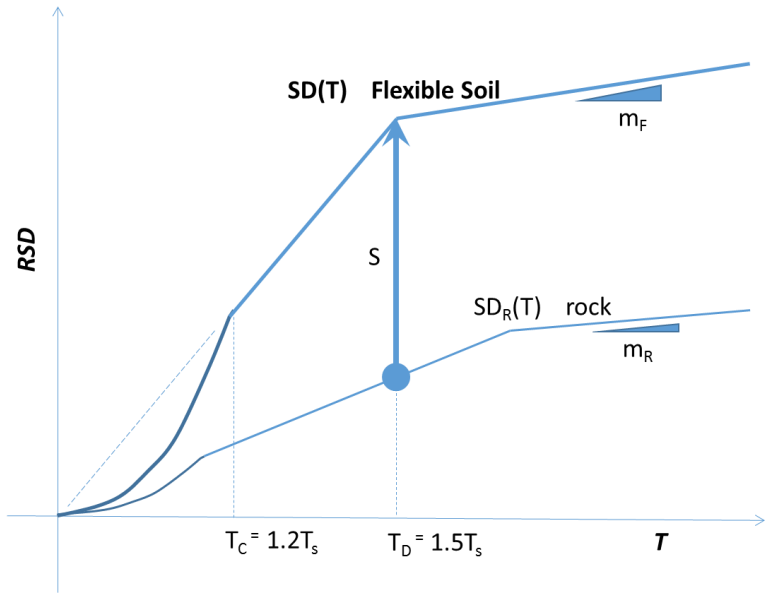
3. Structural performance criteria and parameters for design seismic actions

3.1 Performance Criteria

According to Eurocode 8 – Part 1 (CEN 2004) building structures shall be designed and constructed in such a way that the requirements of (i) *No Collapse* (NC) and (ii) *Damage Limitations* (DL) be met. The state of "no collapse" is essentially in alignment with designing to the *ultimate limit state* which entails the protection of life in a rare earthquake event by ensuring that no parts of the structure collapses and that adequate residual lateral resistant capacity remains in the structure after the event to withstand strong aftershocks should they occur. The safety of occupants of the building can be assured but the built facility can be inhabitable and the damage can be too costly to repair. The "no collapse", or "no local collapse", design criterion as described is comparable to the "life safety" performance criterion as defined in *SEAOC vision 2000* document (SEAOC 1995) in the United States and the "significant damage" (SD) performance criterion stipulated in Eurocode 8 – Part 3 which contains provisions for the seismic assessment and retrofitting of existing buildings. The "no collapse" performance criterion is not to be confused with the "near collapse", or "collapse prevention", performance criterion of *SEAOC vision 2000* which is about ensuring that the building is able to sustain sufficient vertical load carrying capacity in a very rare earthquake event when the structure is on the verge of wholesale collapse with little, or no, residual lateral resistance, and some falling hazards may be present (Fardis 2009).



(a) Stiff Soil Sites (T_S = 0.15s – 0.5s)



(b) Flexible Soil Sites (Ts > 0.5s)

Fig. 2: Schematic representation of Displacement Spectrum Model for Soil Sites

The “Damage Limitation” (DL) performance criterion which corresponds to the serviceability limit state criterion (in the conventional limit state design approach) has also been written into both Part 1 and Part 3 of *Eurocode 8* and is intended to address the damaging potentials of frequent, or occasional, earthquake events in the design of ordinary buildings. The DL performance criterion is comparable to the “immediate occupancy”, or “operational”, performance criterion of *SEAOC vision 2000* which is to

ensure no permanent drift and no loss of lateral strength, and stiffness, of the building structure. The built facility is then fit for continuous occupation in the recovery period and the functionality of the building will not be interrupted significantly by repair activities. In regions of low or moderate seismicity that are remote from tectonic plate boundaries only rare, or very rare, earthquake events are of concern. Thus, the DL performance criterion need not be checked in such environment except for built facilities forming part of *lifeline* facilities in the aftermath of an earthquake disaster or buildings containing hazardous materials. Refer Table 1 for a summary of the performance criteria of building structures as defined by the two parts of Eurocode 8 and the SEAOC vision 2000 document.

3.2 Parameters for design seismic actions

In this section, recommendations for the value of the return period of the design seismic actions, design PGA values for buildings of different importance classes and the behaviour factor are discussed. The return period of the design seismic actions that are aligned with the *no collapse* (NC) performance criterion is to be decided on a country-by-country basis given that factors governing such a decision would involve social, economic and political considerations. Thus, the design return period for the NC performance criterion is to be specified in the respective National Annex of the country.

It is stated in the footnote attached to Clause 2.1 in *Eurocode 8* – Part 1 that ground motion intensity in a rare earthquake event consistent with a 10% chance of exceedance for a design life of 50 years (i.e. Return Period of 475 years) is recommended as the design seismic action. It is noted that this recommendation was drafted in the late 1990's at a time when it was still the norm not to consider return periods exceeding 500 years in the design of structures supporting ordinary buildings. Implicit in the NC performance criterion is that the building is expected to have sufficient additional reserve capacity to sustain a very rare, and extreme, earthquake event without experiencing wholesale collapse (Fardis 2009).

Seismic design provisions around the world have been evolving over many decades during which time experience gained through field observations from places like California have been taken into account in numerous code revisions. In such an environment which is dominated by active faults the intensity of ground shaking is increased by a factor which is slightly greater than 1.5 as the return period is increased from 500 years to 2500 years (Tsang 2014). Code compliant constructions that have been designed to fulfil NC performance criterion is expected to have sufficient additional reserve capacity to also fulfil collapse prevention criterion when subject to seismic actions that are 1.5 times the design level. Despite this margin of safety from collapse that are implicit in contemporary practices, major earthquake disasters occurring in recent years including the 1995 Kobe earthquake in Japan and the 2008 Sichuan earthquake in China prompted a critical review of the adequacy of this long established convention of designing to a return period of 500 years (Tsang 2011).

In regions of low or moderate seismicity (where earthquakes occur infrequently and active faults are difficult to identify) ground shaking intensity ratio that is associated

with an increase of return period from 500 years to 2500 years can be escalated to a value much greater than 1.5. A factor varying between 2.4 and 5 is predicted for earthquakes in an intraplate environment (Tsang 2014; Geoscience Australia 2012). Given these predictions building structures that have been designed to a return period of 500 years to fulfil NC performance criterion in an intraplate environment would not automatically possess adequate additional reserve capacity to prevent collapse in a very rare event.

TABLE 1 Performance Criteria of Building Structures

Eurocode 8 part 1	Eurocode 8 part 3	SEAOC Vision 2000	Descriptions
		Fully Operational	Components that are sensitive to drift and/or acceleration remains fully functional in a **frequent** event.
Damage Limitation (DL)	Damage Limitation	Operational Or Immediate Occupation	No permanent drift and no loss of lateral strength or stiffness of the building. The built facility remains to be fit for continuous occupation in an **occasional** event.
No Collapse (NC)	Significant Damage	Life Safe	No part of the structure collapses and adequate residual lateral resistant capacity remains in the structure after a **rare** event to withstand strong aftershocks in order that safety of the occupants can be secured but building may be inhabitable and repair too costly.
	Near Collapse	Collapse Prevention or Near Collapse	Structure is able to sustain sufficient vertical load carrying capacity in a **very rare** earthquake event when the structure is at the edge of wholesale collapse. Residual lateral resistant capacity of the building might have been lost.

The trend of moving away from the conventional practice of designing to a return period of 500 years was initiated by the influential FEMA450 document (BCCS 2003) which was to guide the design of new buildings in the United States. The design seismic action was recommended to be based on a maximum considered earthquake (MCE) of 2500 years scaled down by a factor of 2/3 (reciprocal of 1.5). This scaling factor can be interpreted as the margin between the state of NC and collapse prevention of the structure in order that code compliant buildings can always be assured of their capacity to prevent collapse in a very rare earthquake event.

The 2005 edition of the *National Building Code of Canada* (NRCC 2005) has increased the return period from 500 years to 2500 years without applying a scaled

down factor of 2/3 (Mitchell 2010) but a generous 2.5% drift limit which is consistent with the *Collapse Prevention* performance criterion has been specified. The National Annex to *Eurocode 8* for the United Kingdom (BSI 2008) also specifies a return period of 2500 years to override the recommendation of 500 years in *Eurocode 8* – Part 1 (CEN 2004) for designing to *No Collapse* (*Life Safe*) performance criterion which is more stringent than requirements in Canada. In perspectives, a design return period of 2500 years is actually not overly conservative given that the annual fatality risk of an occupant in a building which has been designed to a return period of 2500 years is of the order of 10^{-6} which is consistent with involuntary fatality risk affecting building occupants in other types of natural disasters (Tsang 2014; Tsang and Wenzel 2016).

In view of the facts presented in the above design seismic actions presented in terms of design PGA values on rock sites are recommended herein for various importance classes of buildings as summarised in Table 2. It is shown that all built facilities of importance class IV including hospitals, emergency services and other lifeline facilities are to be designed to to a return period of 2500 years to fulfil NC performance criterion in order that lifeline facilities are safe to occupy in the aftermath of a very rare event whilst fit to continue to operate in more frequent events. The design seismic actions for ordinary buildings of importance class II is accordingly based on a PGA value of 0.15g (being 0.22g/1.5) which provides adequate protection of these buildings from collapse in a very rare earthquake event. By interpolation a design PGA of 0.18g is stipulated for buildings of intermediate class III such as condominium, schools and public buildings which can house a large number of occupants at times. This latter class of buildings represent the bulk of the building stock in Hong Kong.

The response spectrum to be used for design purposes for any building class is to be derived from the benchmark model (*HKU DRS 2011, 2015)* based on a return period of 2500 years as presented in Figs. 1(a) – 1(c) and then scaled down in accordance with the respective design PGA value as listed in Table 2. The allowed inter-storey drift limit is 1.5% to fulfil NC, or life safe, performance criterion.

Finally, a behaviour factor (q) is to be stipulated to take into account the capacity of the structure at the member level to withstand seismic actions beyond its notional capacity limits. Design actions (such as bending moments and shear forces) are to be scaled down by the factor of $1/q$ whereas no scaling factor is to be applied in the calculation of drifts, or deformation, in the structure.

In the Australian Standard (AS1170.4 2007) the additional capacity to withstand seismic actions is resolved into the performance factor (S_p) which takes into account contributions from the over-strength of materials and the structural system as a whole in sustaining earthquake generated lateral forces whereas the ductility ratio (μ) takes into account contributions from the ability of the structure to deform in a ductile manner (AEES 2009). The value of S_p is taken by default as 0.77 and the value of μ is taken as 2.0 by default for limited ductile reinforced concrete, structural steel or composite structures which employ concrete and steel as construction materials. The composite factor of 2.6 (being μ/S_p or 2/0.77) that are used as default design value in Australia

can be compared to a slightly lower, more conservative, *q* value of 2.0 recommended in the *National Building Code of Canada* since its 2005 edition (NRCC 2005). The default *q* value stipulated in the National Annex for Singapore is 1.5 which is consistent with recommendations by Eurocode 8. The default values of q that has been stipulated in regulatory documents in various countries of low to moderate seismicity for limited ductile structures are listed in Table 3 to facilitate decision making for Hong Kong.

TABLE 2 Design PGA on rock sites for Hong Kong

Importance Class	Importance Factor	Descriptions	Design PGA (g's)
I	0.8	Minor constructions	0.12 (~0.8 x 0.22/1.5)
II	1.0	Ordinary buildings (individual dwellings or shops in low rise buildings)	0.15 (~0.22/1.5)
III	1.2	Buildings of large occupancies (condominiums, shopping centres, schools and public buildings)	0.18 (~1.2 x 0.22/1.5)
IV	1.5	Lifeline built facilities (hospitals, emergency services, power plants and communication facilities)	0.22 (consistent with RP of 2500 years)

TABLE 3 Default values of *behaviour factor* for limited ductile structures

Region/Country	Standards/Codes	Over-strength factor	Ductility factor	Behaviour factor
Europe Singapore	Eurocode 8 NA to SS	1.5	1.0	1.5
Canada	NBCC	1.3	1.5	2.0
Australia	AS1170.5	1.3	2.0	2.6

4. Conclusions

I. Various response spectrum models that have been developed for Hong Kong on rock by The University of Hong Kong and ARUP have been presented.
II. *Lifeline* facilities including hospitals and infrastructure in support of emergency services are to be designed to fulfil "no collapse" (life safe) performance criterion for a return period of 2500 years. Lower design seismic actions are recommended for buildings of other importance classes.
III. Response spectrum to be used for design purposes is scaled in accordance with design peak ground acceleration values on rock sites which vary between 0.12g and 0.22g. Exact values as presented in the tables depend on the importance

classification of the building.

IV. Schematic representations of response spectrum models for soil sites are also shown.

V. Design actions at the member level such as bending moments and shear forces are to be scaled down by 1/*q* where *q* is the behaviour factor. Values ranging between 1.5 and 2.6 have been suggested.

5. Acknowledgements

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APPENDIX A

The recommended DS model for reference (rock) sites in Hong Kong is expressed in Equation (A1) in the format of response spectral displacement (*RSD*) versus natural period of structure (*T*) (in s), and correspond to a return period of 2475 years.

$$\begin{aligned} T \le 0.23 \quad &: \quad RSD_T(mm) = 0.56 \times (T/2\pi)^2 \times 9810 \\ 0.23 < T \le 1.0 \quad &: \quad RSD_T(mm) = 32 \times T \\ 1.0 < T \le 5.0 \quad &: \quad RSD_T(mm) = 32 + 14 \times (T-1) \end{aligned} \tag{A1}$$

The compatible acceleration DS model can be conveniently obtained by direct transformation of the displacement DS model using Equation (A2).

$$RSD_T(mm) = RSA_T(g)\left(\frac{T}{2\pi}\right)^2 \times 9810 \tag{A2}$$

The variation of the properties of crustal rocks in Hong Kong is relatively confined (Chandler 2006), sub-division of rock categories is not needed. Hence, a single DS model is recommended for the reference rock sites, based on the average crustal rock properties in Hong Kong as characterised in Chandler (2006).