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Recommended seismic performance requirements for building structures in Hong Kong

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Abstract. This paper provides recommendations for setting performance requirements for the seismic design of building structures in Hong Kong. Fundamental issues relating to the required level of structural safety will be addressed, which is then followed with a recommended seismic action model for structural design purposes in Hong Kong. The choice of suitable performance criteria of structures and the return period of the design seismic actions are first discussed. The development of the seismic hazard model for Hong Kong is then reviewed. The determination of the design response spectrum and the choice of design parameters for structures of different importance classes will also be presented.

Keywords: seismic design; building structures; performance requirements; response spectrum; return period

1. Introduction

Seismic design provisions for bridges in Hong Kong have been around for a long time. Meanwhile, Hong Kong does not have an official regulatory document which defines minimum design requirements for building structures in relation to the consideration of seismic hazard. Although Hong Kong is located 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. The first seismic design code for building structures in Hong Kong is currently under development.

Research into the seismic hazard of Hong Kong commenced in the early 1990's. Probabilistic seismic hazard analysis (PSHA) led by ARUP and The University of Hong Kong, respectively, in recent years culminated in the development of Uniform Hazard Spectrum (UHS) models that can be simplified and 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 was aimed at providing recommendations on key decisions that need to be made when drafting the first edition of the seismic design standard for Hong Kong. This paper contains recommendations for improvements over conventional codification practices in regions of low to moderate seismicity. Justifications for these recommendations were evolved from research by the author and/or his collaborators that has continued for over two decades.

General performance objectives and decisions over the choice of the design return period for seismic actions will be discussed first. This is then followed by a review of the literature in relation to the development of the seismic hazard model for the region. The design peak ground acceleration (PGA) values and the corresponding design response spectrum models for the respective return period levels are then provided.

Eurocode 8 (CEN 2004) is currently used as one of the key references to guide the drafting of the future seismic design 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 EN 1998-1 2013) and Malaysia (NA to MS EN 1998-1 2018).

2. Performance criteria and design return periods

2.1 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" (NC) is essentially in alignment with designing to the Ultimate Limit State (ULS) 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 uninhabitable 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

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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 (DL)	Operational or Immediate Occupancy	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 (SD)	Life Safety	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 uninhabitable and repair may be 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.

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” performance criterion in SEAOC Vision 2000, nor with the “Collapse Prevention” level as defined in FEMA Publication 273 (ATC 1997), 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).

The “Damage Limitation” (DL) performance criterion which corresponds to the Serviceability Limit State (SLS) 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 as defined in 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 must also 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 an environment except for more important 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.

In the past decade, there have been attempts to incorporate the amount of risk and loss as a quantifiable measure for defining performance objectives in seismic design. The first risk-based seismic provision was stipulated in the 2010 edition of the structural design standard ASCE/SEI 7-10 and the 2012 edition of the International Building Code (IBC 2012) (adopted principally in the United States). It is required that the collapse risk of ordinary building has to be limited to 1% in 50 years (i.e., an annual probability of exceedance of 2×10^{-4}). However, Porter (2014) pointed out that the rationale behind such a risk limit is not well justified, except that it would not result in too significant changes to the design seismic hazard levels across the whole United States.

There has also been similar development in Europe. Dolšek *et al*. (2017) have proposed a decision model that contains important parameters for risk-based seismic design of buildings, which will be used for guiding the future revision to Eurocode 8. It is noteworthy that Dolšek *et al*. (2017) have adopted a more stringent collapse risk limit of 10^{-4} as recommended by Tsang and Wenzel (2016). Stipulating this risk limit would be sufficient for limiting the individual annual fatality risk of 10^{-6}, which is a tolerable level that has been recommended by various organizations and well supported by historical mortality data that is associated with natural hazard events.

2.2 Design return periods

In this section, recommendations over the choice of the design return period of seismic actions are discussed. The return period of the design seismic actions that are aligned with the “No Collapse” 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 “No Collapse” performance criterion is to be specified in the respective National Annex of the country. The recommendations provided in below are considered reasonable for the well-developed parts of the world.

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 (around 500 years), is recommended as the design seismic action. It is noted that this recommendation was drafted in the 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 "No Collapse" 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 to 2500 years (Tsang 2014). Code compliant constructions that have been designed to fulfil "No Collapse" 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, the 2008 Wenchuan earthquake in China and the 2011 Christchurch earthquake in New Zealand 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 to 2500 years can be escalated to a value much greater than 1.5. A factor varying between 2.4 and 5 is estimated for intraplate environment (Tsang 2014, Lam *et al*. 2016, Geoscience Australia 2012). Given these predictions building structures that have been designed to a return period of 500 years to fulfil "No Collapse" performance criterion in an intraplate environment would not automatically possess adequate additional reserve capacity to prevent collapse in a very rare event.

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 2475 years (around 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 "No Collapse" and "Collapse Prevention" of structure such that code compliant buildings can be assured of their capacity to prevent collapse in a very rare earthquake event. However, a recent study showed that this level of design action may still lead to intolerable levels of annualized individual fatality risk (Tsang *et al*. 2017c) based on the collapse probability estimated by Haselton and Deierlein (2007), Liel and Deierlein (2008), Tsang *et al*. (2016), Hashemi *et al*. (2017).

The 2005 edition of the National Building Code of Canada (NRCC 2005) has increased the design return period from 500 to 2500 years without applying a scaled down factor of 2/3 (Mitchell *et al*. 2010) but a generous 2.5% drift limit which is consistent with the "Collapse Prevention" performance criterion has been specified. In perspectives, a design return period of 2500 years is actually not overly conservative given that the individual annual fatality risk of an occupant in a building which has been designed to a return period of 2500 years can possibly be reduced to a tolerable level in the order of 10^{-6}. Having said that, it is considered pragmatic to adopt the "two-third" scaled down approach as in IBC, and particularly so for the first time when seismic design provisions are enforced in Hong Kong.

2.3 More stringent than wind design?

According to the current Code of Practice on Wind Effects in Hong Kong (BD 2004), structures are required to design to a 50-year return period wind load, together with a load factor of 1.4. At first glance, one may raise a query over the huge difference between the "design return period" for earthquake and wind actions, which seems to indicate a much more stringent performance requirement for countering earthquake hazard. In fact, the reality is the opposite because the level of life safety in an extreme typhoon scenario is actually substantially higher.

Based on an updated analysis of extreme wind speeds in Hong Kong conducted by Holmes *et al*. (2009) (which is intended to be used for the next edition of the Code), a new relationship between the wind speed and return period has been developed. It is found that the factored design wind load (i.e. multiplying the 50-year wind load by 1.4), as stipulated by the Code, is equivalent to the level of wind load with return period of around 1000 years for low-to-medium-rise buildings without significant dynamic effects (i.e., static) and 660 years for high-rise buildings with dynamic effects.

It is noteworthy that a return period of 500 years (for low-to-medium-rise buildings) and 1000 years (for high-rise buildings), with a load factor of 1.0, has been adopted in Australia and New Zealand (AS/NZS 1170.2 2011). Meanwhile, a return period of 700 years has been adopted in the United States (ASCE/SEI 7-10) (coupled with a load factor of 1.0). Hence, the expected performance of structures for wind loads in Hong Kong is similar to that in Australia, New Zealand and the United States. It is reminded that linear elastic response of structure is still expected under these levels of wind loads.

For comparison, wind loads of longer return periods are normalised by the factored design load, and the corresponding "load ratio" curves for "static" and "dynamic" structure are plotted in Fig. 1. It is seen that wind load actually saturates quickly with increase in return period. With the consideration of inherent strength of materials and material partial factor of safety, the typical expected "strength factor" of structure is in the order of 1.5. As shown in Fig. 1, a typical structure is expected to respond in the linear elastic range without any damage up to a wind load with return period of over 5000 years. Clearly, a very high level of life safety can be provided if the building structure is designed conforming to the codes.

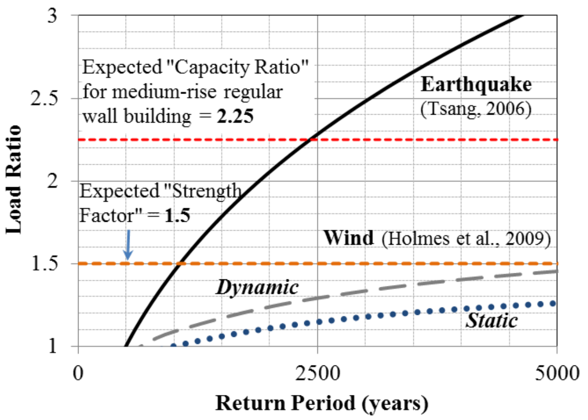


Fig. 1 Comparison of the expected capacity of structure and the design loads of different return periods in Hong Kong

For earthquake actions in Hong Kong, the load demands of various return periods (in terms of response spectral acceleration at 0.2 s) (Tsang 2006) are normalised to that of a return period of 500 years. It is shown that the load increases rapidly without showing a sign of saturation even up to a return period of 5000 years, at which the "load ratio" is greater than 3.0. Meanwhile, with the consideration of the expected "strength factor" of 1.5, the structural response can remain linear elastic up to a 1000-year earthquake action.

Taking a medium-rise regular wall building as example, a nominal ductility capacity of 1.5 can be assumed, which could further contribute to a wider margin against damage and failure. The expected "capacity ratio" of 2.25 can then be used to satisfy the no-collapse requirement under the 2500-year earthquake action. Hence, the recommended performance requirement for seismic design is essentially less stringent than that for wind design in Hong Kong. The values adopted above are only for illustrative purpose. An overstrength factor of 1.5 and a ductility factor of 1.5 are not always available.

3. Design seismic action models and parameters

3.1 Design response spectrum model for rock sites

Research into the seismic hazard affecting Hong Kong dates back to the early 1990's (e.g., Pun and Ambraseys 1992, Lee *et al*. 1998). In those early investigations, the ground motion prediction equations (also known as attenuation models) employed in seismic hazard analysis were mainly based on relationships developed from empirical database of ground motion records collected in the high seismicity region of North America (e.g., California) and Europe (e.g., Greece). Few evidences were available to support the argument that those relationships were representative of local conditions in the South China region. Ground motion properties were represented simply by the PGA of the earthquake. Response spectral parameters were not calculated in those early studies, partly because suitable prediction equations for spectral parameters were not available.

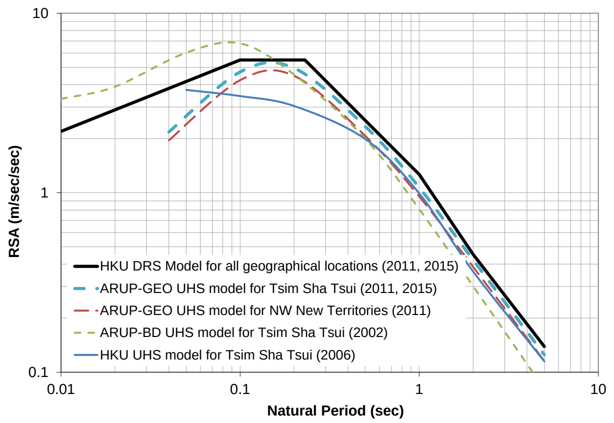
More recent investigations including that by Free *et al*. (2004) took into account distinct features of ground motion and response spectral attenuation properties in the more stable regions of low and moderate seismicity such as Central and Eastern North America (CENA) and many parts of China including 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 at the earthquake source and the wave transmission properties of the earth crust (refer Lam *et al*. 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 program (e.g., 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 *et al*. 2002, Chandler and Lam 2002, Tsang 2006, Chandler *et al*. 2005, 2006a, 2006b). Similar modelling techniques were adapted for developing attenuation relationships for other parts of China (Tsang *et al*. 2010) and many other parts of the world including Australia, India, Iran, Malaysia and Singapore (e.g., Lam *et al*. 2003, 2006, 2009, Yaghmaei-Sabegh and Lam 2010, Chandler and Lam 2004). Importantly, intensity attenuation relationships developed based on site reconnaissance data collected from historical earthquake events in these regions have been reconciled with predictions by the respective seismological model.

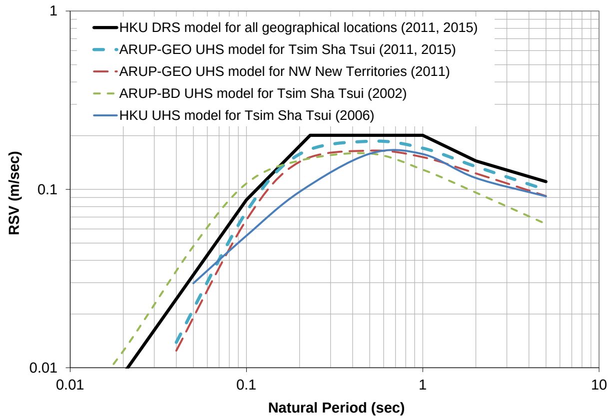
The anelastic attenuation model of Mak *et al*. (2004) was specifically developed to model crustal properties surrounding Hong Kong based on analyses of data from local seismological monitoring. Artificial accelerograms generated based on this locally developed seismological model using program GENQKE enabled ground motion prediction equations to be developed as there was a lack of locally recorded accelerograms (Pappin *et al*. 2008). This ground motion prediction equation has been reported in the literature (Pappin *et al*. 2015a) as the "ARUP–HKU attenuation relationship (2006)" acknowledging joint efforts between ARUP and The University of Hong Kong in collaboration with The University of Melbourne where program GENQKE was written.

Since the 1990's, the concept of the Uniform Hazard Spectrum (UHS) has become common practice for constructing design response spectrum. A UHS incorporates hazards contributed from all potential seismic sources surrounding the site. Normally, short-period spectral values of the UHS are attributed to near-source moderate earthquakes, whereas long-period spectral values reflects the potential hazard from distant larger magnitude earthquakes (Tsang 2015). UHS provides response spectral ordinates for a range of oscillator periods based on PSHA. The advantage of adopting a UHS is that it has a uniform probability of exceedance at all structural periods, and hence, reflects the site-specific frequency content more accurately.

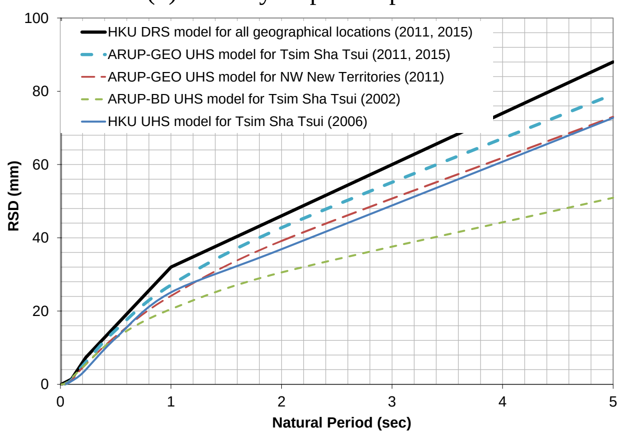
The development of UHS models for Hong Kong has been dominated by the work of two research groups. The first research group is The University of Hong Kong and its



(a) Acceleration response spectrum



(b) Velocity response spectrum



(c) Displacement response spectrum

Fig. 2 Uniform Hazard Spectra (UHS) from various studies and the recommended Design Response Spectrum (DRS) model for rock sites in Hong Kon

UHS (HKU UHS) model was first published in Tsang (2006), and was also presented in Tsang *et al.* (2009), Tsang and Lam (2010) based on the location of the Hong Kong Observatory at 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. Implementing the DAB approach would not require detailed characterization of the seismic sources thereby avoiding uncertainties that are associated with the seismic sources (Tsang *et al*. 2011).

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 model reported by ARUP (2002) is denoted herein as the ARUP-BD UHS model whereas recommendations reported by ARUP (2011) the ARUP-GEO UHS model. In the study undertaken in 2011, ARUP and GEO were in collaboration with the Earthquake Administration of Guangdong Province in China to make recommendations over seismic hazards for the Northwest New Territories of Hong Kong. Seismic hazard modelling for other parts of Hong Kong was also undertaken during that time. Results are presented in the form of seismic hazard contour maps in ARUP (2011), which was subsequently published in Pappin *et al*. (2015a, 2015b). The conventional PSHA methodology, which was pioneered by Cornell (1968), coded into a FORTRAN program by McGuire (1976), and subsequently completed with a deaggregation approach by McGuire (1995), has been adopted in studies undertaken by ARUP.

Good consistencies between the independently developed HKU UHS and the ARUP-GEO UHS models in the intermediate, and high, natural period range can be seen in Figs. 2(a)-2(c). Both studies made use of the ARUP – HKU (2006) attenuation relationship which was underpinned by stochastic simulations of the seismological model (which can be implemented by using 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 in PSHA by the two groups only resulted in minor discrepancies between their recommendations.

Literature references for every item on the legend in Figs. 2(a)-2(c) are listed as follows:

HKU DRS model for all geographical locations (2011, 2015)

– Su *et al*. (2011 & 2015)

ARUP–GEO UHS model for Tsim Sha Tsui (2011, 2015)

– Report by ARUP (2011) and Pappin *et al*. (2015a;b)

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 *et al*. (2009) & Tsang and Lam (2010)

As the hazard levels reported in the ARUP-GEO UHS model vary by 20-30% across Hong Kong (given the different distances from the potential earthquake sources surrounding Hong Kong), a Design Response Spectrum (HKU DRS) model was derived to take into account spatial variations of seismic hazard within Hong Kong (Su *et al*. 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 Figs. 2(a)-2(c) is recommended for codification for Hong Kong (refer Appendix A for details). Given a maximum response spectral acceleration (i.e., highest ordinate of the spectrum in Fig. 2(a)) of 5.5 m/s^2, or 0.56 g, at a return period of 2500 years, the corresponding PGA value is 0.225 g approximately.

The recommended 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. The model is based on the theoretical model published in Tsang *et al*. (2006a, 2006b, 2012) and is in accordance with the response spectral shape as proposed in Lam *et al*. (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 value of scaling parameter S which characterises amplification on the velocity controlled region of the response spectrum is recommended to be 2.5 for stiff soil sites (Tsang *et al*. 2017a) and 3.6 for flexible soil sites (Tsang *et al*. 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.

3.2 Parameters for design seismic actions

Performance requirement varies with the type of structures. Structures are typically categorised into four classes in codes of practice based on the nature of occupancy, which defines the intended use of the structure and the anticipated occupant load. Higher reliability should be provided either because of the consequences of damage or because the structure needs to remain operational during, and after, an earthquake event. Recommendations for the design PGA values for buildings of different importance classes and the behaviour factor are discussed in this section.

The classification schemes in major codes of practice are broadly consistent with each other. 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 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 0.15 g (being 0.22 g/1.5) which provides adequate protection of these buildings from collapse in a very rare earthquake event. By interpolation a design PGA of 0.18 g 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 represents 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. 2(a)-2(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 "No Collapse", or "Life Safety", performance criterion.

Table 2 Recommended values of design peak ground acceleration (PGA) for structures of different importance classes on rock sites in Hong Kong

Importance Class	Importance Factor	Descriptions	Design PGA (g's)
I	0.8	Minor constructions	0.12 (=0.8× 0.225/1.5)
II	1.0	Ordinary buildings (individual dwellings or shops in low rise buildings)	0.15 (=0.225/1.5)
III	1.2	Buildings of large occupancies (condominiums, shopping centres, schools and public buildings)	0.18 (=1.2× 0.225/1.5)
IV	1.5	Lifeline built facilities (hospitals, emergency services, power plants and communication facilities)	0.225 (consistent with return period of 2500 years)

Table 3 Default values of behaviour factor (q) for limited ductile structures

Region/ Country	Standards/ Codes	Over-strength factor	Ductility factor	Behaviour factor
Europe Singapore	Eurocode 8 NA to SS	-	-	1.5
Canada	NBCC	1.3	1.5	2.0
Australia	AS1170.4	1.3	2.0	2.6

Finally, a behaviour factor (q) can 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 in the seismic design of buildings in Hong Kong.

4. Conclusions

This paper discusses the fundamental and influential issues that are associated with the performance requirements and action models for use in the seismic design of building structures in Hong Kong.

I. A return period of around 2500 years, i.e., 2% probability of exceedance in a design life of 50 years is recommended as the "Collapse Prevention" (CP) limit for performance assessment of ordinary structures.

II. Lifeline facilities including hospitals and infrastructure in support of emergency services are to be designed to fulfil "No Collapse" (NC) or "Life Safety" (LS) performance criterion for a return period of 2500 years. Lower design seismic actions are recommended for buildings of other importance classes.

III. Structures are expected to respond in the linear elastic range to a wind load with return period of 5000 years. Hence, higher level of life safety is expected in an extreme typhoon scenario than in a rare earthquake event.

IV. Various response spectrum models that have been developed for rock sites in Hong Kong by The University of Hong Kong and ARUP have been reviewed.

V. Response spectrum models to be used for design purposes are scaled in accordance with the design peak ground acceleration (PGA) values on rock sites which vary between 0.12 g and 0.225 g.

VI. 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.

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Appendix A

The recommended design response spectrum (DRS) model for reference (rock) sites in Hong Kong as expressed in Equation (A1) is presented in the format of response spectral displacement (*RSD*) (in mm) versus natural period of the structure (*T*) (in s), and is based on a return period of 2475 years (around 2500 years). This HKU DRS model mimics the shape of the actual UHS very well.

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

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

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

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

Alternatively, a format consistent with the default model in Eurocode 8 is preferred. Eq. (A3) defines the response spectrum in the acceleration format (in g).

$$\begin{aligned} T \le 0.1:&\quad RSA_T = 0.225 \times (1 + 15 \times T) \\ 0.1 \le T \le 0.25:&\quad RSA_T = 0.225 \times 2.5 \\ 0.25 \le T \le 2.0:&\quad RSA_T = 0.225 \times 2.5 \times \left(\frac{0.25}{T}\right) \\ 2.0 \le T \le 5.0:&\quad RSA_T = 0.225 \times 2.5 \times \left(\frac{0.5}{T^2}\right) \end{aligned} \tag{A3}$$

The alternative DRS model as defined by Eq. (A3) features a constant spectral displacement region in the long period range (i.e., T higher than 2.0 s). The spectral demand is slightly overestimated for T in the range 1.0 s-3.5 s, and underestimated for T exceeding 3.5 s, in comparison with the actual UHS or the original DRS model of Equation (A1). The second corner period (i.e., the lower period limit of the constant displacement region) can be increased to avoid understating seismic actions in high period structures.