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Cursory seismic drift assessment for buildings in moderate seismicity regions

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Abstract: This paper outlines a methodology to assess the seismic drift of reinforced concrete buildings with limited structural and geotechnical information. Based on the latest and the most advanced research on predicting potential near-field and far field earthquakes affecting Hong Kong, the engineering response spectra for both rock and soil sites are derived. A new step-by-step procedure for displacement-based seismic hazard assessment of building structures is proposed to determine the maximum inter-storey drift demand for reinforced concrete buildings. The primary information required for this assessment is only the depth of the soft soil above bedrock and the height of the building. This procedure is further extended to assess the maximum chord rotation angle demand for the coupling beam of coupled shear wall or frame wall structures, which may be very critical when subjected to earthquake forces. An example is provided to illustrate calibration of the assessment procedure by using actual engineering structural models.

Keywords: seismic hazard assessment; response spectrum; soil site; rock site; displacement-based; inter-storey drift; coupling beams; chord rotation angle

1 Introduction

Over the last decade, an important advance in earthquake engineering has been the elaboration of performance-based concepts for the seismic design of structures. To achieve satisfactory building performance through design or to evaluate an existing building, one needs to reconcile expected seismic demands with acceptable performance levels while recognizing the uncertainties involved. Lateral drifts are the main cause of structural damage in buildings subjected to earthquake ground motions. Additionally, lateral drifts are also responsible for earthquake-induced damage to many types of nonstructural components in buildings. During preliminary design of new buildings or for a cursory seismic evaluation of existing buildings, there is a need to estimate the maximum lateral displacements that can occur in the building when subjected to earthquake ground motions with various probabilities of occurrence. The inter-storey drift angle, which is associated with the upper limit of column drift demand, is particularly relevant for checking the structural stability of multi-storey buildings with weak column - strong beam arrangements to prevent the development of soft-storey mechanism. Inter-storey drifts also concern building facades, vertical piping, lifts and other precise building services equipment in taller buildings. It has been shown in recent earthquake events that damage to such elements accounts for more than 50% of the repair bill (Brunsdon, 2001). Failures of facades in tall buildings in a congested urban environment can also cause injuries and deaths, as well as costly disruptions to the continuous function of facilities.

A number of performance-based earthquake engineering procedures exist, including the so-called “walk-through” simplified assessment procedures, whereby buildings are qualitatively assessed for seismic resistance using a systematic but relatively qualitative form-based procedure, that can be highly subjective and unreliable when transported to regions other than those intended (primarily western USA). More elaborate and versatile procedures are included in the US code planning document, ATC-40 (1996) “Seismic Evaluation and Retrofit of Concrete Buildings.” According to ATC 40, performance-based earthquake design has two key

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elements, namely the evaluation of the capacity and demand of the structure and the resulting elements displacement. The alternative "Direct Displacement-Based (DB) Assessment" procedure introduced in recent years by Priestley and co-researchers was developed in the US and New Zealand (Priestley, 1995). The primary intention of both of these procedures is to check whether the available strength and post-elastic displacement capacity of the building matches the requirements of active seismic regions. Yun *et al*. (2002) presented a procedure for seismic performance evaluation of steel moment frames, based on nonlinear dynamics and reliability theory. It is, however, only relevant for steel structures. Chandler *et al.* (2002b) introduced a preliminary procedure for drift-based seismic assessment of buildings. However, the degree of deformation of the coupling beams has not been quantified in this study. Hidalgo *et al*. (2002) presented an analytical model including both flexural and shear failure modes, which was integrated into a computer program for predicting the reinforced concrete shear wall structure displacement performance.

There are also many methods presented by researchers for predicting the displacement capacity or demand of a structure. Paulay (2002) used bi-linear modelling of force displacement relationships for reinforced concrete components or systems, which is generally accepted as being adequate for the purposes of seismic design, to predict the displacement capacity of reinforced concrete coupled walls. The virtue of this method is that the estimation of displacement capacities of components of a system enables the critical components to be identified and gives designers preliminary knowledge for the designed structure. Mohammadi (2002) derived an empirical formula for approximate deflection amplification factors in predicting the seismic displacement capacity of structures. The empirical formula for this code-based factor was related to the allowable ductility ratio, the fundamental period and the number of storeys. However, the above two methods are useful for performance-based seismic design but not for performance-based assessment. Chopra and Goel (2002) developed an improved pushover analysis procedure to estimate the seismic demands of buildings based on structural dynamics theory. Gupta *et al*. (2000) outlined a process for the estimation of seismic roof and storey drift demands for frame structures from the spectral displacement demand at the first mode period of the structure by a series of modification factors. This study, however, only consider the first mode effect.

To assess the seismic hazard of buildings, the overall dynamic performance of a building has to be determined and the degree of local deformations of some of the critical structure elements, such as coupling beams, has to be quantified. It is well known that coupled shear walls or core walls are commonly employed as the major lateral load-resisting system in tall building structures. Coupling beams are required to connect the wall piers and transfer loading between them. Under seismic action, the shear and deformation capacities of coupling beams in coupled shear wall or core wall systems are often critical. Failure of coupling beams may lead to more serious subsequent failure of the whole lateral load resisting system of the building. The deformation of coupling beams may be quantified by the chord rotational angle. Adebar and White (2002) presented a formula for estimating the maximum coupling beam chord rotation demands in high-rise coupled wall buildings. The formula for chord rotation of relatively regular buildings related with wall slope and floor slope was derived from the results of non-linear dynamic analyses of relatively regular structures. Lam *et al.* (2003) studied the deformation and ductility capacity of common and steel plate composite coupling beams. Harries (2001) reviewed large-scale experimental investigations of coupling beam behaviour. It demonstrated that the coupling beam ductility demand often exceeded the expected available ductility. Thus it is critical for studying the performance of coupling beams under seismic loading. In this study, the deformability of coupling beams is characterized by the chord rotation angle.

Of necessity, general methods for obtaining the displacement capacity and demand of structure and elements are the nonlinear static analysis (pushover analysis) and nonlinear dynamic time history analysis that can track the development of ductile mechanisms and hence predict the overall seismic behaviours. However, the non-linear static push over analysis or dynamic analysis requires a detailed computer model of the building. The computer model will often have to be developed from scratch since the electronic copy of the structural analysis is usually no longer available, and the majority of pre-70's and 80's designs for normal projects are supported only by manual calculations. Thus, the ATC-40 procedure is costly on a per-building basis. Consequently, a relatively small sample of buildings can be analysed while the majority of buildings are scanned by some sampling criteria. In 1995, Priestly established displacement-based approaches for seismic assessment of buildings, in which the overall drift demand at an effective height of building (or at roof level) is compared with the corresponding drift capacity curve in order to assess the degree of damage as well as the factor of safety of the building. This approach cannot, however, distinguish whether the drift is uniformly distributed at each storey or is concentrated at a particular storey that may be caused, for example, by soft-storey or higher mode effects. The other approaches based on computer structural model analysis are also time-consuming and costly.

This paper presents a relatively simple and inexpensive procedure to quickly estimate the maximum demand of the inter-storey drift angle for general reinforced concrete buildings. This procedure is also extended to predict the maximum demand of chord rotation angle of coupling beams for shear wall or core wall structures. The prediction formula for maximum inter-storey drift demand of buildings has considered the higher mode effects. This method is established based on the results of the earthquake response spectra method and standard statistical methods. It is noted that, although the displacement spectra are defined for elastic response conditions, they may equally represent the displacement demand of inelastic medium to long-period systems according to the well-known *equal displacement rule* (Chopra, 2001; Sun, *et al.* 2004). On this basis, the results obtained and the displacement prediction method may be generalized to medium and tall buildings responding moderately into the inelastic range. For the latter conditions, however, an allowance for increased levels of effective damping maybe required depending on the expected level of ductility demand. The primary information required for this assessment comprises the depth of soft soil above bedrock, the height of the building as well as the span-depth ratio of the coupling beams, if any for the building concerned. The simplicity of the procedure means that a much larger sample of buildings is objectively assessed for the given time and resources. Furthermore, effective sampling criteria can be developed relatively easily.

The procedure enables the determination of the maximum storey drift in multi-story buildings, and the chord rotation angle of coupling beams in shear wall structures. This proposed procedure has the important attribute of not requiring the lateral stiffness of the building to be estimated (as in the case of a force-based calculation procedure) and has been calibrated by sixteen building models (as shown in Table 3) and seismic data in Hong Kong. It is noted, in particular, that none of the buildings considered in this study has significant vertical irregularities such as the presence of soft-stories. It is also important to note that the modelling methodology was developed from linear elastic dynamic and static analyses of the building models considered. Distortion resulting from shear shift was not involved in the study. The methodology is therefore more suitable to the performance-based assessment of buildings in elastic or nearly elastic range when subjected to earthquake ground shaking in regions of low-to-moderate seismicity. The predicted maximum drift demand provides a good indication of the likelihood of damage occurrence as well as the extent of damage, despite the apparent limitations of linear elastic analyses.

2 Seismic hazard assessment in Hong Kong

2.1 Path and site effects modelling

The magnitude-dependent source properties $\alpha(M)$ of the component attenuation model (CAM) have been dealt with by Lam and Chandler (2002). The attenuation of seismic waves is significantly more regionally dependent than source properties; thus, it is inappropriate to propose a generalized relationship. Ideally, attenuation is best modelled by correlating locally recorded strong motion with distance, using a large representative database. A direct approach is often not achievable, due to the chronic lack of strong-motion data in regions of low to moderate seismicity such as South China (including Hong Kong). An alternative viable approach is to estimate the effects of individual wave modification mechanisms based on geophysical parameters, which can be determined without strong motion records.

The seismic response parameters can be predicted by combining the source factor, the path component factors and the site factor as described in this section, using the following expression:

$$Y = \alpha(M)\cdot G(R,D)\cdot \beta(R,Q)\cdot \gamma_{mc}(V_{source})\cdot \gamma_{uc}\cdot S(T_s, V_{sc}) \tag{1}$$

where the response parameter Y is either D_{RSmax} or V_{RSmax}, V_{RS} indicates the period-dependent response spectral velocity, and D_{RS} is the corresponding response spectral displacement, and D_{RSmax} and V_{RSmax} are their maxmum values respectively. Equation (1) in the CAM model initially was developed in Lam *et al.* (2000a,b). The attenuation relationships are for predictions of the mean.

The combined path effects (which exclude local site modifications and will be dealt with below) have been expressed as the product of the following component factors: (i) geometrical attenuation factor $G(R,D)$, (ii) anelastic attenuation, or energy absorption, factor $\beta(R,Q)$, (iii) the mid-crustal amplification factor $\gamma_{mc}(V_{source})$, and (iv) the upper crust modification factor γ_{uc}; where the associated parameters are the crustal depth (D, in km), the shear wave velocity gradient of the earth's crust and the wave transmission quality factor (Q). The component factors as summarized in Lam and Chandler (2002) were derived by curve-fitting results from stochastic simulations of regional seismological parameters.

Shaking from distant earthquakes can also induce very significant soil amplification effects, as a result of the robust transmission of high period energy (in the velocity and displacement-sensitive parts of the response spectrum) over long distances. The 1985 Mexico City and 1989 Loma Prieta, California earthquakes offered well-known recent examples of this phenomenon. The site response component factors briefly described in Table 1 have been developed based on the "frame analogy soil amplification" (FASA) model, as described by Lam *et al*. (2001). The term V_{sc} is the shear wave velocity of the bedrock (m/s). The basis of FASA is that the amplitude of the soil spectrum (soil V_{RSmax}) can be obtained by scaling from V_{RSmax} of the corresponding rock spectrum at the natural period of the site (T_s). This modelling concept is supported by results obtained from non-linear wave analysis and by field observations. Soil

Table 1 Site factor $S(T_s, V_{sc}) = \mu\alpha\lambda\tau$

Demand parameters	μ	α	λ	τ
D_{RSmax}	1.2	4-6	$0.65+\frac{V_{sc}}{10000}\le 1.0$	$\frac{T_s}{T_2}\le 1$
V_{RSmax}	1.2	4-6	$0.65+\frac{V_{sc}}{10000}\le 1.0$	1

resonance, which is a key issue for Hong Kong (Sheikh, 2001) cannot be ignored for buildings without provision of ductile seismic design in regions of moderate seismicity. Resonance must be fully accounted for if the structure is liable to undergo little energy dissipation prior to damage (e.g., damage to building facades and piping) or instability (e.g., a soft storey mechanism).

The μ−factor, which accounts for velocity and displacement amplification up the soil column, is estimated to be ~1.2 based on a parabolic displacement profile. The α−factor, which accounts for the resonance of the above-ground single degree of freedom (SDOF) structure, varies between 4 and 6 for a soil shear strain ~0.1%-0.2% (representative of low to moderate seismic conditions). The λ−factor is needed to account for the effect of radiation damping, which increases with decreasing bedrock shear wave velocity V_{sc}. A λ−value of 0.7 is recommended for young ("soft") sedimentary rocks with V_{sc}~500 m/s, whereas an upper bound λ−value of 1.0 is recommended for hard crystalline volcanic or metamorphic rocks, or hard glaciated sedimentary rocks. In summary, the predicted overall soil amplification factor (the product $\mu\alpha\lambda$) typically varies between around 3.5 (i.e., 1.2×4×0.7) and 7 (i.e., 1.2×6×1.0), depending mainly on the hardness of the underlying bedrock and to a lesser extent on the soil plasticity and the intensity of the excitation. A nominal 5% of critical damping for the above-ground structure has been assumed.

The τ−factor models the important difference in displacement amplification between high period (deep) and low period (shallow) soil sites (Fig.1). This effect has not been parameterized in design codes, which typically classify sites according to average soil properties (average shear wave velocity) rather than the soil depth. For this reason, the same amplification has been specified for both the velocity-sensitive and the displacement-sensitive regions of the response spectrum in code models.

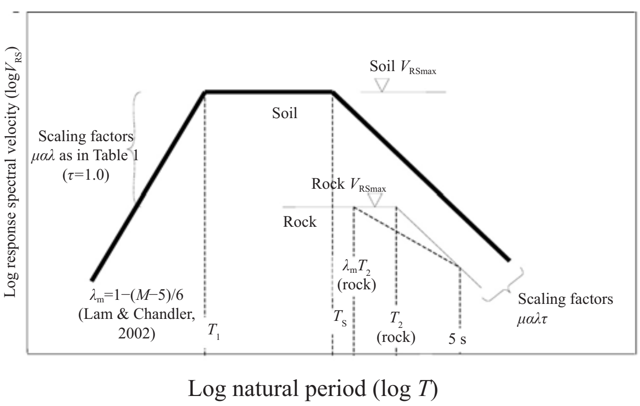


Fig. 1 Soil and rock response spectra modelled by CAM and FASA procedures

2.2 Procedure for displacement-based seismic hazard assessment

The seismic hazard is appropriately expressed in terms of the engineering response spectrum (ERS) for rock and soil sites in Hong Kong. Such ERS have been developed for various design return periods, referred to the probability of exceedance (PE) of the hazard in a 50-year design exposure period. Herein, the PE has been taken as 50%, 10%, 5% and 2%, relating, respectively, to design return periods of 72, 475, 975 and 2475 years. Table 2 summarizes some of the critical seismic hazard data used in the subsequent drift angle estimations. The purpose herein is to provide a methodology for determining the seismic displacement demand on the building, in terms of $D_{RS}(T_1) = D_{RS1}$, where T_1 is the building's fundamental lateral natural period in the direction of loading, as discussed below. The following step-by-step procedure is proposed and should start from Step 2 for soil sites, or at Step 1 for exposed rock sites or where the soil depth is found to be less than around 3–5m.

Step 1. Establish the rock D_{RS} spectrum (in mm) as a tri-linear form (Lam *et al.*, 2002), as follows:

For T_1= 0 - $\lambda_m T_C$:
$$D_{RS} = \frac{T_1}{2\pi} V_{RS\max} \tag{2}$$

For T_1=$\lambda_m T_C$ -5 s:

$$D_{RS} = \frac{\lambda_m T_C}{2\pi} V_{RSmax} + \left(D_{RSmax} - \frac{\lambda_m T_C}{2\pi} V_{RSmax} \right)\left(\frac{T_1 - \lambda_m T_C}{5 - \lambda_m T_C} \right) \tag{3}$$

For T_1 >5 s :
$$D_{RS} = D_{RSmax} \tag{4}$$

where

$$\lambda_m = 1 - \frac{M-5}{6} \tag{5}$$

Table 2 Seismic hazard data from LAM *et al*. (2002) and CHANDLER *et al*. (2002)

Seismic parameter	50% PE/ 50 years	10% PE/ 50 years	5% PE/ 50 years	2% PE/ 50 years
Design earthquake magnitude (M) for critical far-field events at site-source distance R=280km	6.7	7.4	7.6	7.8
Corner period on rock displacement response spectrum (D_{RS}), T_C (s)	2.1	2.3	2.35	2.35
$V_{RS}(T_i)=V_{RSmax}$ for Rock in mm/s T_i = initial site period in the range 0.2-2.0 s for soil sites	34	70	84	100
V_{RSmax} for Soil in mm/sec for $T_1 < T_g$ T_g = seismic site period (sec)	220	400	460	540

also

$$D_{RSmax} = \frac{T_C}{2\pi} V_{RSmax} \tag{6}$$

For rock sites, go to Step 7.

Step 2. Calculate soil depth, H (m). Refer to Chandler & Su (2000), for a definition of characteristic depth H; alternatively define H as the depth to materials with shear wave velocity V_s>500 m/s.

Step 3. Calculate weighted average shear wave velocity V_s (m/s) over depth H. Refer to Chandler & Su (2000).

Step 4. Determine the initial site period T_i(s)= $4H/V_s$

Step 5. Determine the seismic site period T_g (s)=ηT_i where the period shift ratio η = 1.1 + 0.003$V_{RS}(T_i)$ and $V_{RS}(T_i)$ is given in Table 2. Hence η = 1.2–1.4 for the range of PE considered. Refer to Chandler *et al*. (2002c).

Step 6. Establish the soil D_{RS} spectrum (in mm) as a bi-linear form, as follows:

$$\text{For } T_1 = 0 - T_g: \quad D_{RS} = \frac{T_1}{2\pi} V_{RSmax} \tag{7}$$

$$\text{For } T_1 > T_g: \quad D_{RS} = D_{RSmax} \tag{8}$$

where

$$D_{RSmax} = \frac{T_g}{2\pi} V_{RSmax} \tag{9}$$

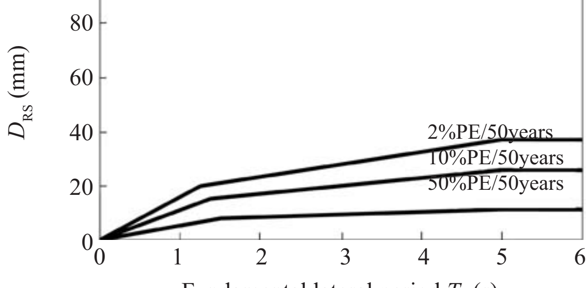
and V_{RSmax} is as given in Table 2. Refer to Chandler *et al*. (2002c).

Step 7. Estimate the building's fundamental lateral period, T_1 (s), refer Eq.(24) below.

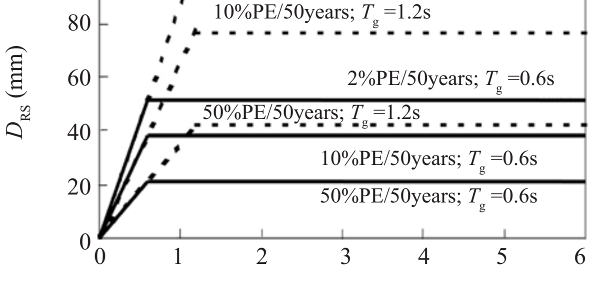
Step 8. Determine $D_{RS}(T_1) = D_{RS1}$ in mm, from the rock or soil D_{RS} response spectrum, as defined in Step 1 or Step 6, respectively.

Step 9. The above displacement response spectra assumed a standard structural damping ratio ζ_b of 5%. In cases where different damping ratios ζ (in %) are more appropriate, the D_{RS} for both rock and soil sites should be modified by the factor $\sqrt{10/(5+\zeta)}$.

Because Hong Kong is such a large city, in this study the collapse prevention level for building performance was chosen to be conservative. According to ATC 40, collapse prevention is required for ground motions with a 2500 years return period (2% PE in 50 years). Therefore, the V_{RS} and D_{RS} response spectra with PE equal to 2% in 50 years on soil and rock sites in HK, which were developed according the seismic hazard assessment of the Hong Kong region (Chandler & Lam, 2002; Lam *et al*., 2002; Chandler *et al*., 2002a), have been used. Using the above procedures, D_{RS} for rock sites and soil sites have been evaluated and presented in Fig. 2.



(a) Rock sites



(b) Soil sites

Fig. 2 Displacement response spectra D_{RS}

3 Definitions of seismic drift factors for buildings

3.1 Drift factors for SDOF buildings

The maximum seismic inter-storey drift angle $\theta_{1\max}$ can be related to the effective displacement D_{RS1} (see previous section) using the following generic expression:

$$\theta_{1\max} = \lambda_{1\max} \frac{D_{RS1}}{H_b} \tag{10}$$

where H_b is the building height and $\lambda_{1\max}$ is the maximum dynamic drift factor to be determined by the response spectrum analyses. The maximum inter-storey drift angle $\theta_{1\max}$ could be correlated to the average drift angle θ_{avg} $(= \Delta_{1roof}/H_b)$ by the drift factor λ_1 as

$$\lambda_1 = \frac{\theta_{1\max}}{\theta_{avg}} \tag{11}$$

Furthermore, the lateral displacement at the roof level Δ_{1roof} is related with D_{RS1} by the factor λ_{avg}

$$\lambda_{avg} = \frac{\Delta_{1roof}}{D_{RS1}} \tag{12}$$

By substituting Eqs.(11) and (12) into Eq.(10), it is found that

$$\lambda_{1\max} = \lambda_1 \lambda_{avg} \tag{13}$$

After obtaining $\lambda_{1\max}$, the maximum inter-story drift angle for SDOF buildings can be obtained from Eq. (10).

3.2 Drift factors for multiple degree of freedom (MDOF) buildings

For MDOF buildings, the maximum seismic inter-storey drift angle $\theta_{\max}$ (with consideration of higher vibration modes) may be related with the maximum inter-storey drift angle $\theta_{1\max}$ (as defined in section 3.1.) associated with the first lateral vibration mode $\boldsymbol{\Phi}_1$. The ratio between these two drift angles is denoted as the higher mode drift factor λ_2 such that

$$\lambda_2 = \frac{\theta_{\max}}{\theta_{1\max}} \tag{14}$$

hence,

$$\theta_{\max} = \lambda_1 \lambda_2 \lambda_{avg} \frac{D_{RS1}}{H_b} = \lambda_{\max} \frac{D_{RS1}}{H_b} \tag{15}$$

defining

$$\lambda_{\max} = \lambda_1 \lambda_2 \lambda_{avg} \tag{16}$$

$\lambda_{\max}$ is the maximum dynamic drift factor to be determined by the response spectrum analyses, which can be considered to consist of the higher mode effect (λ_2) and the fundamental mode effects (λ_1 and λ_{avg}). The definitions of the various drift angles are illustrated in Fig. 3(a).

The local deformations of coupling beams characterized by the chord angle θ_b (defined in Fig. 3(b)) are related to the predicted maximum inter-storey drift demand as well as the span-to-depth ratio of the beam. After determining $\lambda_{\max}$, the maximum inter-storey drift angle $\theta_{\max}$ can be obtained readily from Eq. (15). For the case of quasi-static vibration, which can be justified for medium to high-rise buildings of which the induced internal loads of coupling beams are controlled by the response spectral displacement D_{RS}, the maximum chord angle $\theta_{b\max}$ of the coupling beams in the building can be shown to be related to both the span-to-depth ratio of the beam and the inter-storey drift angle $\theta_{\max}$ by a chord angle factor λ_b as:

$$\theta_{b\max} = \lambda_b \theta_{\max} \left(L/d\right)^2 \tag{17}$$

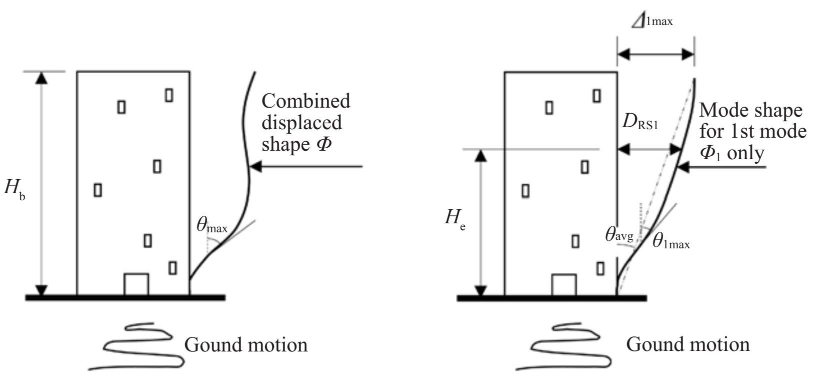
where d is the depth of the beam and L=2L_1 is the equivalent span of the coupling beam as defined in Fig. 3(b). L_1 is the distance measured from the support of the beam with the higher bending moment to the point of contra-flexure. It is noted that the new definition of effective span may or may not be equal to the clear span. The purpose of adopting this definition is to simplify the complicated boundary conditions of coupling beams (fixed ends, fixed and pinned ends on each supports, etc.) to the equivalent fixed end conditions such that the point of contra-flexure is always located at the middle of the equivalent span. For coupling beams firmly fixed onto identical wall panels on both ends, the point of contra-flexure will be at the mid-span and the equivalent span will be equal to the clear span. Otherwise, the equivalent span will be longer than the clear span. For the case of fixed support at one end and pinned support at the other end of the beam, the equivalent span will be twice that of the clear span.

All the drift factors λ_i may be calibrated using dynamic analysis of real buildings with a wide range of heights and types. Based on the free vibration analysis of a building, individual vibration mode shapes $\boldsymbol{\delta}_j$ can be determined. The associated mode shape $\boldsymbol{\Phi}_j$ caused by the seismic displacement spectra, as mentioned in the last section, can be obtained by proper scaling of the mode shape $\boldsymbol{\delta}_j$ with the modal participation factors P_{fj} and D_{RS},

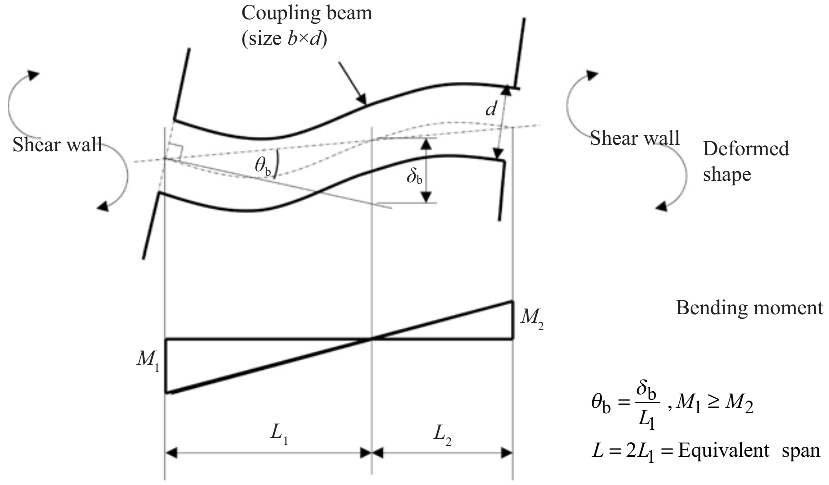
$$\boldsymbol{\Phi}_j = P_{fj} D_{RS}(T_j) \boldsymbol{\delta}_j \tag{18}$$

where

$$P_{fj} = \left(\frac{\sum_{i=1}^{n} m_i \delta_i}{\sum_{i=1}^{n} m_i \delta_i^2} \right)_j \tag{19}$$



(a) Drift angles θ_{max}, θ_{1max} and θ_{avg} of a building under seismic load



(b) Chord rotation angle θ_b of a coupling beam.

Fig. 3 Illustrations for various drift angles

and m_i is the mass at the ith floor.

The lateral roof displacement of the first vibration mode Δ_{1roof} can be determined from the corresponding mode shape $\boldsymbol{\Phi}_1$ at the roof level. Hence, the average drift angle θ_{avg} (= Δ_{1roof}/H_b) can be obtained. The maximum inter-storey drift angle θ_{1max} of the first vibration mode can be expressed as

$$\theta_{1\max} = P_{f1}\max\left(\frac{\delta_i - \delta_{i-1}}{h_i}\right)_1 D_{RS}(T_1) \qquad (20)$$

where h_i is the floor height at the ith floor. By using the square root of the sum of the squares method (SRSS) to combine all the modal responses and considering the first three lateral vibration modes in a given principal direction, the maximum inter-storey drift of the building θ_{max}, with consideration of higher mode effects, may be expressed as

$$\theta_{\max} = \max\left(\sqrt{\theta_{1i}^2 + \theta_{2i}^2 + \theta_{3i}^2}\right) \qquad (i\text{=1 to } n) \qquad (21)$$

where n is the number of storeys and

$$\theta_{ji} = P_{fj}\left(\frac{\delta_i - \delta_{i-1}}{h_i}\right)_j D_{RS}(T_j),\ j = 1 \text{ to } 3.$$

Having evaluated Δ_{1roof}, θ_{avg}, θ_{1max} and θ_{max} from Eqs. (18) to (21) and D_{RS1} from Eqs. (2) to (9), all the drift factors λ_i can be calculated from Eqs. (11) to (17).

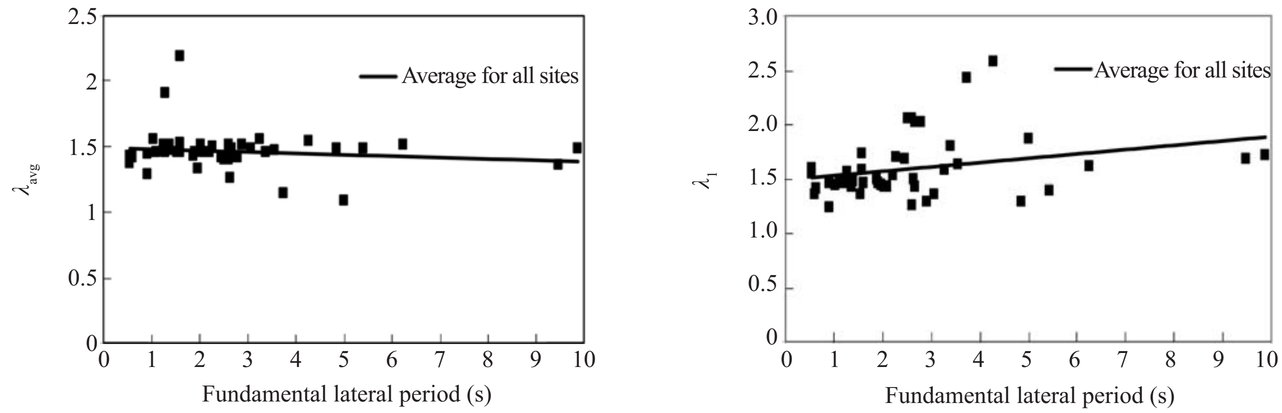
3.3 Calibration of seismic drift factors

Sixteen buildings with general information as shown in Table 3 have been used to calibrate the various drift factors. The height of the buildings considered varies in a wide range from 35m to 430m. Most are reinforced concrete buildings located in Hong Kong. Structural forms are varied from typical moment resistant frames or simple shear walls for low-rise buildings to shear wall systems with or without transfer plates (TP) for high-rise buildings, to core wall and outrigger truss systems for super-high-rise buildings.

Based on dynamic analysis employing the above procedures and the statistical regression method, all the drift factors can be best fitted with lines or curves. Table 4 shows the all drift factor formula.

Table 3 General information of buildings for calibration of the drift factors

Building label	Location	Total height H_b (m)	No. of storey N	Lateral vibration period (s)		Structural form
				Higher	Lower	
CBAB	Beijing	35	10	0.562	--	Shear wall
SK	Hong Kong	50	17	1.253	0.617	Shear wall
TTT1	Hong Kong	53	13	1.962	1.585	Frame and wall
BSB1	Hong Kong	60	13	2.035	1.607	Frame
CMW1	Hong Kong	71	21	2.098	1.912	Shear wall with TP
SLY	Hong Kong	98	37	2.664	2.211	Shear wall
AMC7A	Hong Kong	113	35	3.080	--	Shear wall with TP
THB0	Hong Kong	118	42	2.904	2.615	Shear wall
THB1	Hong Kong	118	42	2.786	2.623	Shear wall
51BM	Hong Kong	126	43	3.275	2.281	Shear wall
CMW2	Hong Kong	132	42	3.558	3.398	Shear wall with TP
BHP	Australia	152	40	3.746	--	Frame, core wall and outrigger trusses
SB	Australia	169	43	4.863	--	Core wall
ALDR	Hong Kong	169	57	4.081	3.845	Shear wall and TP
OB	Australia	306	76	6.250	--	Frame and core wall
WHK	Hong Kong	430	98	9.346	8.856	Frame, core wall and outrigger trusses



(a) λ_{avg} (b) λ_1

Fig. 4 Variation of drift factors

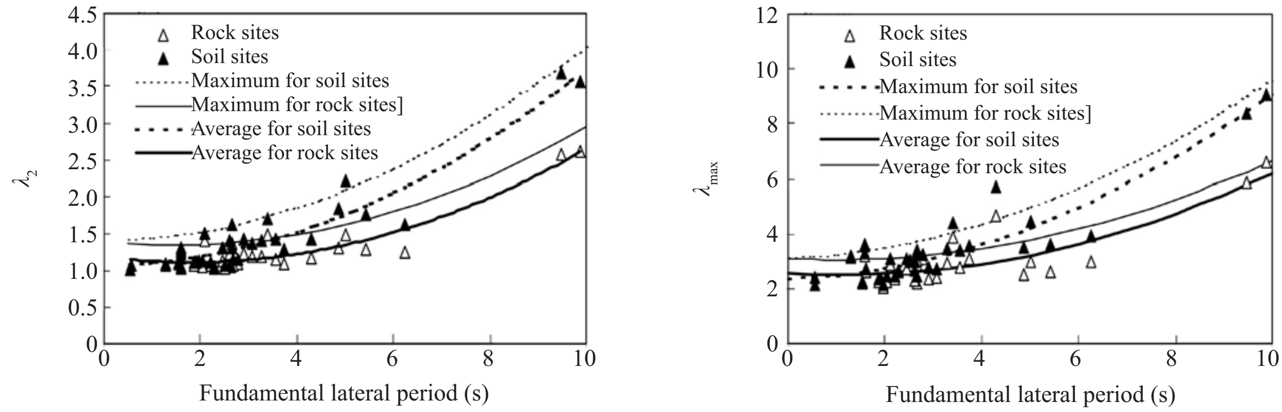
From Fig. 4, and equations for the drift factors of λ_{avg} and λ_1, the drift factors of λ_{avg} and λ_1 are found to be insensitive to building fundamental lateral period T_1. Both factors typically vary from 1.2 to 1.8 with a mean value of 1.5.

The variation of the higher mode factor λ_2 with the building fundamental lateral period T_1 is shown in Fig.5(a).

It is observed that the factor increases from 1.0 for low-rise buildings to around 3.5 for high-rise or super-high-rise buildings for soil sites. The increment of this factor is found to be much slower for rock sites. The average curve equation for λ_2 is formulated by regression analysis. To conservatively represent those variations, a general formula for the upper limit of λ_2 is proposed by incrementing the mean curve equation with constant values of 0.35 and 0.25 for soil and rock sites, respectively.

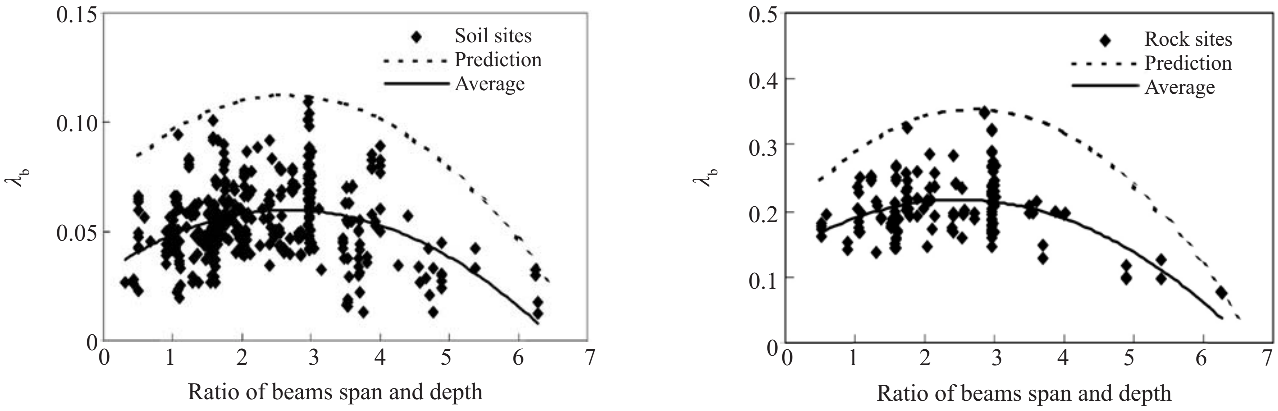
The maximum dynamic drift factors can be derived from Eq. 16. The third order coefficient of T_1 is very small and has been ignored in the equations. The variations of λ_{max} are shown in Fig. 5(b). Reasonable agreement is observed between the best fit curves and the numerical data.

From the dynamic analysis and the above-mentioned procedures, the coupling beam rotational factor can also be found. Figure 6 shows the variation of average rotational factor λ_b and the upper limit rotational factor λ_b with the beam span-to-depth ratio (ρ).



(a) λ_2 (b) λ_{max}

Fig. 5 Variation of drift factors for higher mode



(a) Soil sites (b) Rock sites

Fig. 6 Variation of beam rotation factors

4 Methodology for cursory seismic drift assessment

4.1 Building seismic drift estimation

It is well known that the response spectral displacement D_{RSj}, arising due to an individual mode of vibration j, is a function of modal period T_j, such that

$$D_{RSj} = D_{RS}(T_j) \tag{22}$$

where T_j is a natural lateral vibration period of the building. For j=1, the D_{RS} due to the first fundamental lateral mode is D_{RS1}. The earthquake induced roof displacement Δ_{1roof} has been related with the response spectral displacement as shown in Eq. (12), and λ_{avg} has been given in Table 4. The average drift angle θ_{avg} can be estimated by the following expression

$$\theta_{avg} = \frac{\Delta_{1roof}}{H_b} = \lambda_{avg}\frac{D_{RS1}}{H_b} \tag{23}$$

The seismic induced maximum inter-storey drift can be predicted by Eq.(15), which depends on the maximum dynamic drift factor λ_{max}, D_{RS1} and the fundamental lateral period of building T_1 (or building height H_b). The maximum dynamic drift factor λ_{max} has been calibrated in the previous section and the results are expressed in Table 4 for soil and rock sites. Both D_{RS1} and λ_{max} depend on the building's fundamental lateral period T_1. Su *et al*. (2002) estimated the fundamental lateral period of typical dynamic computer models for buildings in Hong Kong and found the following empirical formula for fundamental lateral period of bare frame (BF) models, herein called the BF model, means that regarding the lateral stiffness, only the contribution of structural elements is considered in design by practicing engineers.

$$T_{bf} = 0.025H_b = \frac{H_b}{40} \tag{24}$$

The general procedure for rapid prediction of θ_{max} and θ_{bmax} has been summarized in Fig. 7. Following the procedure, it is easily to estimate the maximum story drift of a building and the chord rotation angle of the beams in the building.

Table 4 Formula for all drift factors

Factors Name		Soil site	Rock site
λ_{avg}		$-0.011T_1+1.5$	
λ_1		$0.039T_1+1.5$	
λ_2	$\bar{\lambda}_2$	$0.025T_1^2+0.013T_1+1.06$	$0.022T_1^2-0.065T_1+1.15$
	$\tilde{\lambda}_2$	$0.025T_1^2+0.013T_1+1.41$	$0.022T_1^2-0.065T_1+1.40$
λ_{max}	$\bar{\lambda}_{max}$	$0.06\ T_1^2+0.073T_1+2.35$	$0.046T_1^2-0.096T_1+2.55$
	$\tilde{\lambda}_{max}$	$0.056T_1^2+0.088T_1+3.12$	$0.044T_1^2-0.086T_1+3.11$
λ_b	$\bar{\lambda}_b$	$-0.004\rho^2+0.022\rho+0.03$	$-0.013\rho^2+0.064\rho+0.14$
	$\tilde{\lambda}_b$	$-0.006\rho^2+0.032\rho+0.07$	$-0.022\rho^2+0.12\rho+0.19$

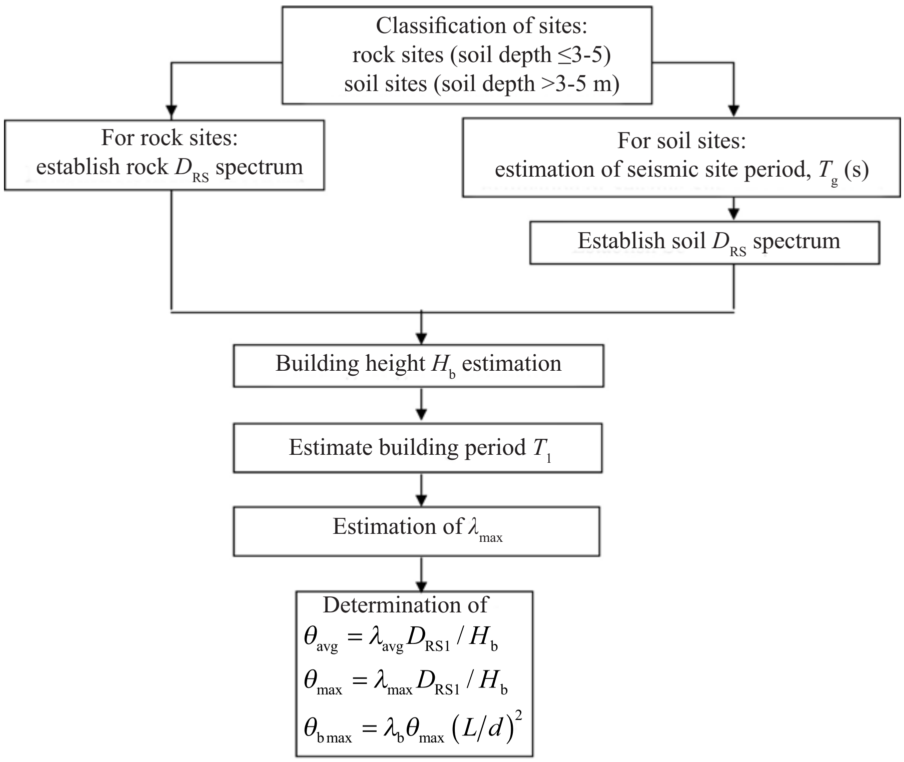
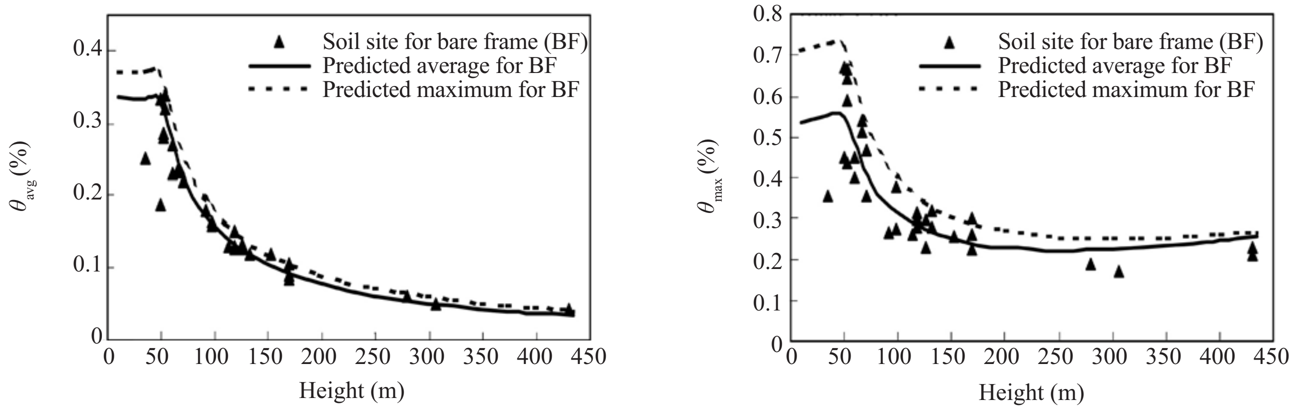


Fig. 7 General procedure for the rapid assessment of seismic drift for buildings

4.2 Evaluation of proposed method

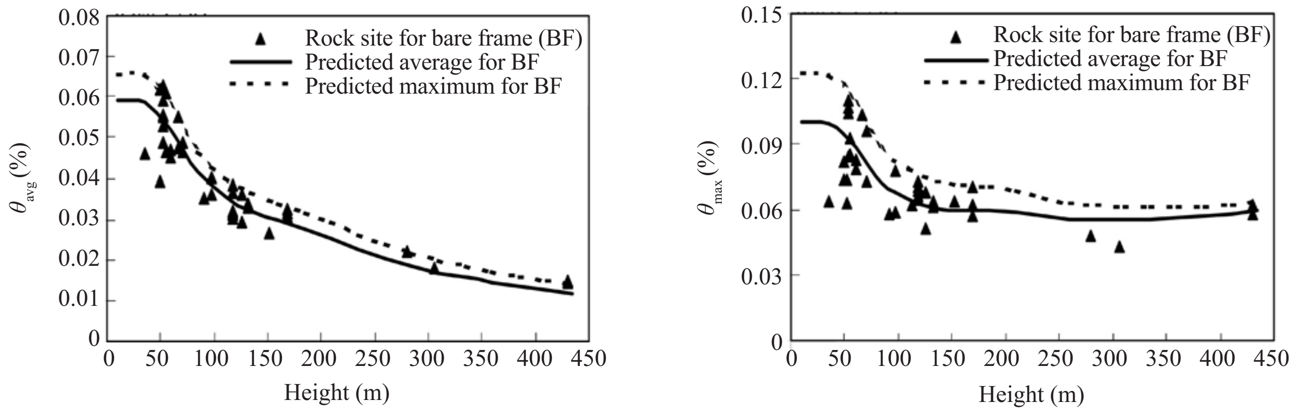
To demonstrate the validation of the proposed procedure for estimating the building drift angle θ_{avg} and maximum inter-storey drift angle θ_{max} for buildings, the predicted values of the drift angles are shown in Figs. 8 and 9 for soil and rock sites, respectively. Here, all the buildings are assumed to be founded on deep soil or rock sites, with seismic action having 2% PE/50 years. Based upon the data base of buildings considered, the maximum inter-story drift angle is found to be 0.72% with a peak at H_b=70m, which is associated with soil sites having T_g=1.2 sec for the bare frame model (BF) building.

The seismic induced maximum chord angle of coupling beams can be predicted by Eq.(11), which depends on the maximum dynamic drift factor λ_b, θ_{max} and the span-to-depth ratio *L/d* of the coupling beam. The predicted maximum coupling beam chord angle θ_{bmax} for BF models are shown in Figs. 10(a) and 10(b) for soil and rock sites, respectively. In these figures, chord angles of coupling beams in buildings with heights 70m, 150m, and 400m have been predicted. As beams under combined high shear stresses and high chord rotation demands are likely to be more vulnerable, only the beams with shear stress higher than 3.5 MPa, which is equal to the maximum allowable working stress in British Standard BS8110, were selected for calibration of the chord rotational factor. It is found that the high shear stress in coupling beams is mainly associated with those beams having span-to-depth ratio below 4. The predicted θ_b, according to Eq. (17), reached



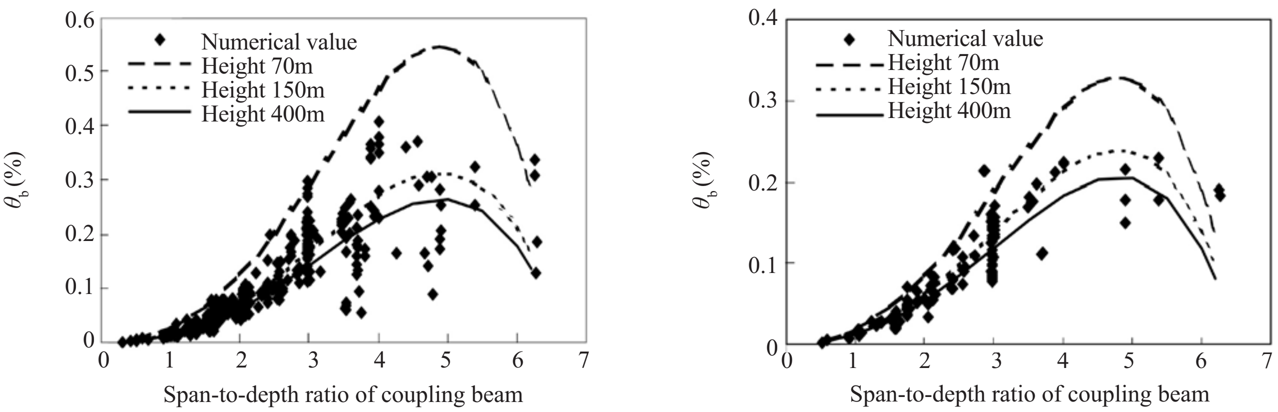
(a) θ_{avg} (b) θ_{max}

Fig. 8 Variation of drift angles against building heights of deep soil site with 2% PE/50years



(a) θ_{avg} (b) θ_{max}

Fig. 9 Variation of drift angles against building heights of deep rock site with 2% PE/50years



(a) Building on soil sites (b) Building on rock sites

Fig. 10 Predicted and numerical results of beam chord angles with different heights of buildings

the highest value when the span-to-depth ratio is about 5.0. However there are only a few coupling beams whose span-to-depth ratio is more than 4 and shear stress exceeds 3.5 MPa. The maximum chord angle demand is found to be 0.4% for BF models on a deep soil site.

5 Conclusions

A simple procedure has been developed to predict the seismic drift demands, accounting for dynamic displacement shape and higher mode effects in reinforced concrete buildings. The principal conclusions arising from this study are summarized as follows.

(1) The proposed procedure indicates that maximum drift demands are much higher for deep soil sites in Hong Kong, compared with rock sites. The maximum inter-storey drift of bare frame (BF) model buildings, far-field earthquakes with 2% PE in 50 years is predicted to be on the order of 0.72% on deep soil sites, when the low damping of Hong Kong buildings is taken into account. In contrast, for rock sites, the maximum inter-storey drift demand ratio has been computed to be only around 0.12%.

(2) Buildings with a fundamental lateral period in the region of 1-2 s (corresponding to building height of around 70–150m or typically 23-50 storeys) are particularly susceptible to large values of seismic inter-storey drift demands, for both soil and rock sites.

(3) The maximum coupling beam chord angle demands of BF models for deep soil sites are much higher than that of rock sites in HK. The maximum coupling beam chord angle due to rare, far-field earthquakes with 2% PE in 50 years is predicted to be on the order of 0.4% on deep soil sites at span-to-depth ratio of 4. In contrast, rock sites are reduced to around 0.25%.

(4) Coupling beam chord angle with span-to-depth ratio higher than 3 is particularly scattered, for both soil and rock sites.

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