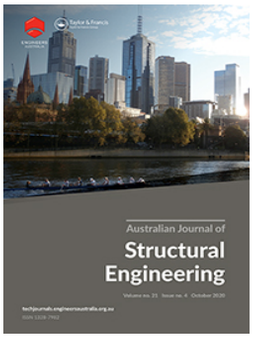

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Collapse probability of soft-storey building in Australia and implications for risk-based seismic design

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ABSTRACT
Collapse prevention is the primary objective of earthquake-resistant design of structures; hence, the probability of collapse should be taken as a crucial performance indicator for risk-based design of new structures or assessment of existing structures. One major challenge in collapse risk assessment is to reliably model the non-linear structural response behaviour. This study features the rocking response behaviour of precast reinforced concrete (RC) columns based on results from previous field testing on parts of a real building and supplemented with a study of their rocking behaviours through a series of shake-table tests. The effects of bidirectional earthquake actions on failure drift capacity of columns have also been incorporated, such that realistic estimates of displacement capacity were made for constructing collapse fragility functions, which were then combined with the ground motion recurrence relationships of Melbourne, Australia for the computation of collapse probability. A suite of typical soft-storey buildings was adopted, with considerations given to a diversity of site conditions. Deaggregation of the results reveals the range of return periods that controls the collapse risk, which could have important implications for the choice of earthquake scenarios for seismic analysis and design in regions of lower seismicity.

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Risk-based Seismic Design; Collapse Probability; Soft-storey Building; Drift Capacity; Return Period

1. Introduction

Although Australia is located in a region of lower seismicity, the earthquake risk should not be ignored as the majority of building structures were not designed in accordance with a seismic code. There were around 10 earthquakes with magnitude greater than 5.0 that have occurred since 1950, notably including the 1968 M_S 6.8 Meckering (WA) earthquake and the 1989 M_L 5.6 Newcastle (NSW) earthquake that took away 13 people lives and injured more than 160. The consequences of a major earthquake that strikes a populated area in Australia could be unbearable, as seismically vulnerable buildings and infrastructure are prevailing. Recent earthquake events of moderate magnitude (e.g. 2011 M_W 6.2 Christchurch earthquake in New Zealand, which led to 185 deaths) demonstrated that actual ground motion can locally exceed the design ground motion significantly (Tsang 2011). It was evident that code-compliant and well-constructed buildings in a well-developed country could still collapse in an earthquake of moderate magnitude (which can also occur in Australia).

There has been an increasing interest globally in incorporating various residual risk measures in the performance objectives of earthquake-resistant design. More details can be found in the recent review by Douglas and Gkimprixis (2018). Based upon the findings in the FEMA P-695 report (prepared by ATC 2009), the 2012 edition of the International Building Code (IBC) and the 2010 edition of the structural design standard ASCE/SEI 7 specify the performance requirement of having uniform collapse risk for structures that are designed based on the risk-targeted maximum considered earthquake (MCE_R) ground motions (with a return period around 2,500 years). Under the MCE_R ground motions, it is expected to have less than 10% probability of collapse for ordinary structures (Risk Category I and II, equivalent to Importance Level 1 and 2 in AS/NZS 1170.0 (2002) and National Construction Code of Australia (NCC) (2019)), 6% for structures that house large number of people (Risk Category III, equivalent to Importance Level 3 in AS/NZS 1170.0 (2002) and NCC (2019)) and 3% for critical facilities (Risk Category IV, equivalent to Importance Level 4 in AS/NZS 1170.0 (2002) and NCC (2019)). IBC (2012) and ASCE/SEI 7–10 also specify a requirement of 1% probability of collapse in 50 years for Risk Category I and II structures, and less than 1% for Risk Category III and IV structures.

On the other side of the Atlantic, risk-targeted seismic design maps have been developed for mainland France (Douglas, Ulrich, and Negulescu 2013) and Romania (Vacareanu et al. 2018), and ongoing studies have been conducted towards comparing various methods (Gkimprixis, Tubaldi, and Douglas 2019),

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formulation of risk-targeted seismic action (Žižmond and Dolšek 2019) and developing a similar risk-targeted map that covers the whole Europe (Silva, Crowley, and Bazzurro 2016). Dolšek (2015) has put forward a set of risk-based performance objectives for seismic design, which is used for guiding the future revision of Eurocode 8 (EN 1998–1 2004). Besides the target collapse risk, the expected amount of economic losses is proposed to be used for controlling the amount of damages due to earthquakes. Dolšek, Sinković, and Žižmond (2017) have proposed a decision model that contains important parameters for risk-based seismic design of buildings and an iterative risk-based structural design procedure has also been put forward (Sinković, Brozovič, and Dolšek 2016). Also, Tsang and Wenzel (2016) have made recommendations for the acceptable level of collapse risk of individual building by controlling fatality risk. Fatality risk was also used as a measure of the consequence for the assessment of seismic performance (Tanner and Hingorani 2015; Crowley et al. 2017; Sinković and Dolšek 2020). Both risk-targeting approach and minimum-cost approach have recently been explored by Gkimprixis, Tubaldi, and Douglas (2020).

The aforementioned design requirements are based on collapse risk of individual buildings. However, there is no indication of the consequence to the whole society. The use of loss assessment for guiding seismic design level has initially been proposed by Bommer and Pinho (2005), which was followed up in an initial attempt by Crowley et al. (2012) to obtain the recurrence relationship for economic loss that can be used for calibrating code level. Societal risk function can also be developed through earthquake scenario-based loss modelling (Tsang et al. 2018a), in terms of economic loss or casualties, which can then be used for evaluating code level (Crowley, Silva, and Martins 2018; Tsang et al. 2020).

In this paper, the collapse risk of a suite of typical soft-storey buildings, with precast reinforced concrete (RC) columns of different dimensions, will be computed and benchmarked against the requirements such as in IBC (2012) and ASCE/SEI 7–10. A wide range of site conditions, using site natural period as a site classification parameter, will be considered. The calculated probability will then be deaggregated, such that the range of return periods that dominates the collapse risk can be discovered. The results presented in this paper can provide evidence and help inform whether seismic retrofitting for soft-storey buildings (e.g. Zabihi et al. 2018; Liu, Tsang, and Wilson 2019; Tsang 2019; Raza et al. 2019a) is justified or not, which is part of a national research programme about developing a cost-effective strategy for mitigating building-related natural hazards risk in Australia (Tsang et al. 2016). Insights relevant to risk-based seismic design will also be drawn towards the end of the paper.

2. Soft-storey buildings

Soft-storey building is one of the most vulnerable types of construction globally, which has been defined as Category 1 vulnerable RC buildings in Australia (Menegon et al. 2019). They are featured by a storey (typically at the ground level) with significantly less lateral load resisting elements, whilst the rest of the building (typically sitting above the soft storey) being stiffer due to the presence of substantial number of load-bearing walls (see Figure 1(a) for an example). When an earthquake strikes, the displacement demand on the structure would mainly be distributed to the soft storey, whereas the upper storeys move together as a rigid box. Hence, the drift capacity of columns at the soft storey become critical to the vulnerability of such buildings. Extensive research has been conducted in recent years on the lateral load and drift capacity of RC columns (e.g. Hashemi et al. 2017; Raza, Tsang, and Wilson 2018; Raza et al. 2019b).

Full-scale field testings on a demolition site of a soft-storey building supported by precast RC columns have been reported in Wibowo et al. (2010, 2011) (Figure 1(b)). Laboratory and analytical investigations have since been extended to soft-storey buildings supported by cast in-situ RC columns (e.g. Raza et al. 2020a). It has been shown that those soft-storey buildings can still perform well at the design life safety (LS) or ultimate limit state (ULS) level, which is corresponding to a notional design return period of 500 years according to AS 1170.4 (2007). When the buildings are subject to unidirectional earthquake actions (Raza et al. 2019b), the buildings can also fulfil the requirement of collapse prevention (CP), which is defined in the FEMA Publication 273 (ATC 1997), or the near collapse (NC) level as defined in the Vision 2000 performance-based seismic design framework (SEAOC 1995) and the Eurocode 8 – Part 3, which are typically associated with a design return period around 2,500 years. However, the level of residual risk in this type of buildings for earthquake hazards beyond the CP or NC level has not been identified nor adequately investigated in previous studies. Also, the effects of realistic bidirectional actions have not been examined either. Hence, this study attempts to address these issues. Figure 1

An analytical model for estimating the lateral displacement capacity of precast RC columns has been derived from first principles using fundamental theories of mechanics (Wibowo et al. 2010, 2011). Estimates obtained from the analytical model have been verified by comparison with results from field

(a)

(b)

Figure 1. (a) A typical soft-storey building supported by precast RC columns in Melbourne, Australia (photo taken by the first author). (b) Set-up of the field test on a demolition site (Wibowo et al., 2010, 2011).

testings. Meanwhile, shaking table testings have been conducted by the authors to study the rocking behaviour of free-standing objects in dynamic conditions (Kafle et al. 2011). The rocking model has since been adapted to modelling the fragility behaviour of soft-storey buildings under unidirectional actions of intraplate earthquakes (Kafle et al. 2015).

It was observed from field tests (Wibowo et al. 2010) that the precast RC column can be displaced significantly, and it tends to behave similarly to a rocking system when the building is subjected to ground shaking. The failure of the columns can, therefore, be defined as overturning of a rigid body. The peak displacement demand (*PDD*) has been found to be the governing ground motion parameter for simulating rocking action and overturning behaviour. It was also revealed that the risk of earthquake-induced overturning decreases predominantly with increasing size of the column, rather than the aspect ratio (Kafle et al. 2015). Hence, the risk assessment will be conducted for columns of four different sizes in this study, in order to evaluate the 'size effects' on the annualised collapse probability.

3. Collapse fragility function

A lognormal cumulative distribution function (CDF) as presented in Equation (1) is used to define the seismic collapse fragility functions for the four precast columns, which predict the probability of collapse (C) due to rocking-induced overturning under unidirectional earthquake actions given a certain level of *PDD* (see Figure 2).

$$P(C|PDD) = \Phi\left(\frac{\ln(PDD) - \ln(\overline{PDD})}{\sigma}\right) \quad (1)$$

where $P(C|PDD)$ is the conditional probability that a ground motion with a certain level of *PDD* will cause the structure to collapse, Φ is the standard normal CDF, $\overline{PDD}$ is the median value of *PDD* that has a 50% chance of structural collapse and σ is the standard deviation of the normal distribution that represents the $\ln(PDD)$ values. Note that the mean of $\ln(PDD)$ is corresponding to the median of *PDD* as *PDD* is lognormally distributed. The mean and

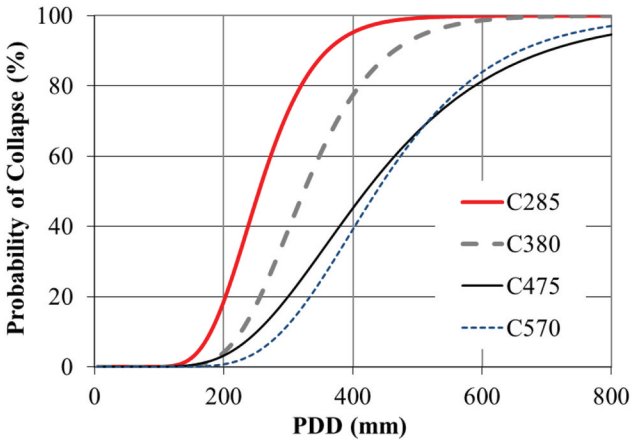


Figure 2. Collapse fragility functions for soft-storey buildings with precast RC columns of different sizes based on peak displacement demand (*PDD*).

standard deviation values for the four columns are summarised in Table 1 (Kafle et al. 2015).

Collapse limit state is defined in the FEMA P-695 report (ATC 2009) as dynamic instability (e.g. overturning) due to excessive lateral deformations of certain components or the estimated demands exceeding a pre-defined limit. With these conditions, nearly complete damages of structure with low residual lateral strength and stiffness must be anticipated. However, wholesale collapse rarely occurs, and the degree or proportion of collapse depends on the robustness of the structure and the intrinsic properties of the construction materials. More discussion can be found in Tsang and Wenzel (2016). This is consistent with the complete structural damage (CSD) state adopted in HAZUS (FEMA 2012), where 3 to 15% of buildings at CSD state would collapse based on historical data. Hence, the definitions of collapse limit state in those documents are roughly consistent with that adopted in this study, for which the values of *PDD* are associated with the limit state of collapse for 5% probability of overturning (Kafle et al. 2015).

Earthquake ground shaking is well known as being irregular and multidirectional. Due to the limitations of experimental facilities, research on the seismic or cyclic behaviour of structures has traditionally been focused on the situation when unidirectional actions are applied. It is acceptable to consider unidirectional actions alone if there are lateral load resisting elements predominantly in one direction of the structure, of which the columns would largely be displaced in the orthogonal direction. However, it is not the case for the building selected for this study. Hence, it is prudent to consider the effects of bidirectional actions (Raza et al. 2020b) and even the variation of axial load (Raza et al. 2020c) on the drift capacity of the columns.

Table 1. Parameters characterising the seismic collapse fragility functions for soft-storey buildings with precast RC columns of different sizes.

Index	Column Size (mm)	$\overline{PDD}$ (mm)	σ
C285	285	255	0.27
C380	380	324	0.28
C475	475	420	0.40
C570	570	437	0.32

A simple biaxial interaction factor has been proposed by Raza et al. (2020b) based on experimental data for adjusting the failure drift capacities of RC columns under significant bidirectional earthquake actions. The median value of *PDD* that has a 50% chance of structural collapse, i.e. $\overline{PDD}$ in Equation (1), can be adjusted in a similar way to $\overline{PDD}_b$ as expressed by Equation (2) whilst the standard deviation σ remains unchanged.

$$\overline{PDD}_b = \frac{\overline{PDD}}{1 + b/a} \tag{2}$$

in which b/a is the ratio of the non-concurrent maximum displacement in the smaller loading direction and the larger loading direction, which was found to be around 0.6 on average (Raza et al. 2020b). In other words, a factor of 0.625 has to be applied to reduce the predicted failure drift of the columns. More results and discussion can be found in Section 5.

Furthermore, it is worth noting that further reduction in drift capacity is expected when variable axial load is considered. Raza et al. (2020c) has found that when the variation of axial load is nonsynchronous to the lateral displacement of the building, which is the likely situation for the structural layout of the case study building, the failure drift capacities would become 25% lower, which would, in turn, increase the collapse risk. However, the variation of axial load is not taken into account in the analysis presented in Section 5.

4. Displacement demand estimates

The computation of annualised collapse probability requires seismic hazard predictions for an annual frequency of exceedance as low as 10^{-5} or sometimes lower than 10^{-6}. The only set of hazard results that provides estimates for annual frequencies down to 2×10^{-5} (i.e. return period of 50,000 years) for Melbourne, Australia, can be found in Somerville et al. (2013), which is therefore adopted in this study. As *PDD* on rock sites can be conveniently defined as the spectral displacement demand at natural period T of 1.5 s, i.e. $S_d(1.5)$, the uniform hazard spectra (UHS) in Somerville et al. (2013) were digitised, and the values at 1.5 s were then fitted with a power function, as shown in Figure 3. Hence, the hazard function in terms of the annual frequency of exceedance of *PDD* for rock sites can be represented by Equation (3):

$$H(PDD) = 0.0002 \times \left(\frac{61.5}{PDD}\right)^{2.7} \tag{3}$$

Strictly speaking, the probability (P) and the average rate of exceedance (λ) within a certain period of time (t_D) are different, which can be related by using Equation (4) based on Poisson's distribution, with an assumption that the number of events occurring in an interval of time depends only on the length of the interval and does not vary in time. The return period is the reciprocal of the annual rate of exceedance. The probability (P) is approximately equal to λt_D if the product of λ and t_D is small (<0.05 in the context here). Hence, the annual probability of collapse, $P(C)$, can be used interchangeably with the annual average rate of collapse, λ_C, in the context of this study.

$$P = 1 - e^{-\lambda t_D} \approx \lambda t_D \tag{4}$$

The case study building with different size of precast soft-storey columns is assumed to be located on a suite of sites covering various soil conditions, with the natural period of the whole soil layer, T_S, equals 0.3 s, 0.5 s, 0.7 s, 1.0 s and 1.2 s. For simplicity, the weighted average shear wave velocity over the whole thickness of all five sites is 240 m/s, whilst the total thickness, H_S, varies from 18 m to 72 m. These sites can be categorised into different site classes (refer Table 2) according to the refined site classification scheme (Tsang, Wilson, and Lam 2017a) recommended for the Australian Standard for Earthquake Actions, AS 1170.4. It is noted that the upper boundary of 0.6 s has already been used for Class C sites in the current edition of AS 1170.4.

The design response spectra for various soil sites were derived based on a theoretical model for estimating resonant-like amplification behaviour (Tsang et al. 2017b). The response spectral amplification factor, S, can be expressed as

$$S = f(\alpha)\frac{2\alpha}{1+\alpha}\sqrt{\frac{\beta}{1-R^4\beta^4}} \tag{5}$$

where α, β, f and R are, respectively, the ratio of seismic impedances between bedrock and soil materials, half-period damping factor, resonance factor (as a function of α) and reflection coefficient as defined in Tsang, Chandler, and Lam (2006a).

On the other hand, the site period lengthening ratio, η, can be expressed as

$$\eta = 1 + R_\gamma \xi \mu \frac{S_d(T_S)}{H_S}\left(\frac{\pi}{2}\right) \tag{6}$$

where μ, ξ and R_γ are, respectively, plasticity factor, a reduction factor that quantifies the effects of radiation damping and the ratio of effective shear strain to maximum shear strain (= 0.6 unless otherwise specified) as defined in Tsang, Chandler, and Lam (2006b).

Equations (5) and (6) can provide estimates of nonlinear spectral amplification factors for different sites and at different shaking levels. For sites with weighted average shear wave velocity of 240 m/s, bedrock shear wave velocity of 1800 m/s (Tsang, Sheikh, and Lam

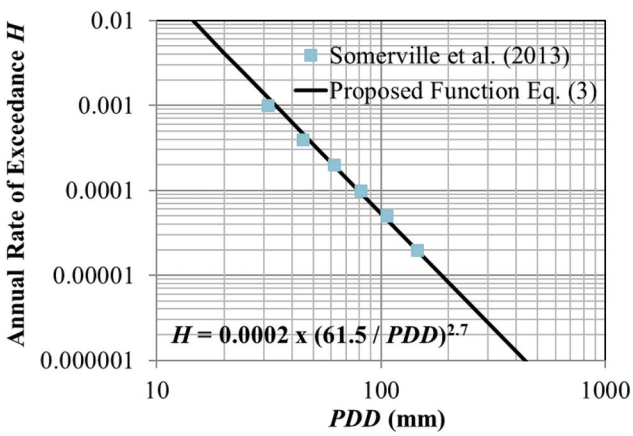


Figure 3. Seismic hazard function, *H*, for Melbourne, Australia in terms of peak displacement demand (*PDD*).

Table 2. Total thickness of soil (H_S) and site natural period (T_S) of each site (weighted average shear wave velocity over the whole thickness is 240 m/s for all five sites), and the corresponding site class based on the refined site classification scheme for AS 1170.4 (Tsang, Wilson, and Lam 2017a).

Site	H_S (m)	T_S (s)	Site Class
1	18	0.3	C
2	30	0.5	C
3	42	0.7	D
4	60	1.0	E
5	72	1.2	E

2012) and taking a shaking level corresponding to a return period of 2,500 years as an example, the site period lengthening ratio is estimated to be around 1.4 and the spectral amplification factor at the shifted site natural period to be around 3.5. Table 3 summarises the second corner period, T_2, and the *PDD* for the five sites with a return period of 2,500 years based on Melbourne conditions. Similar calculations have been done for the whole range of return periods being considered in this study.

5. Collapse probability calculation

5.1. Risk at 2,500-year earthquake action

The collapse risk of a soft-storey building with precast columns subjected to the 2,500-year unidirectional earthquake actions has been calculated for all 20 building-site combinations (i.e. combinations of the four column sizes and the five soil sites). The collapse probability is generally very low, with the highest probability of 6.1% for the smallest column (285 mm) sitting on the most onerous, yet very rare, site with initial site period of 1.2 s. This collapse probability is lower than the stipulated limit of 10% in IBC (2012) and ASCE/SEI 7–10 for Risk Category II structures that include the type of residential buildings being considered in this study. However, the collapse probability would be marginally higher than the 6% limit for Risk Category III structures.

As discussed in Section 3, the failure drift capacities of columns could be significantly impaired when bidirectional displacement demand exists. For column size of 285 mm, earthquake ground shaking with a unidirectional *PDD* of 255 mm would result in a 50% chance of overturning (Table 1). There is still an ample margin from the much lower demand of 157 or 168 mm on a very flexible soil (Class E) site (Table 3), which explains the low collapse probability. However, nearly 40% reduction in the failure drift capacity of columns would be anticipated if there is no lateral restraint in any direction of the building structure. This would bring down the value of $\overline{PDD}$ from 255 mm to 159 mm, which becomes comparable to the demand. In other words, the collapse probability would be increased significantly to around 50% for the most onerous, very rare combination.

For the more typical column size of 380 mm or 475 mm, the collapse probability of the soft-storey building sitting on a very flexible soil site would still be in excess of the 10% limit stipulated in IBC (2012) and ASCE/SEI 7–10 for Risk Category II structures, whilst the risk would become insignificant in the more common situations when the building is sitting on a Class D or a stiffer site. This highlights the importance of local site effects in seismic design.

Table 3. Site natural period (T_S), second corner period (T_2) and peak displacement demand (*PDD*) of the design response spectrum of each site with a return period of 2,500 years.

Site	T_S (s)	T_2 (s)	*PDD* (mm)
1	0.3	0.42	48
2	0.5	0.70	78
3	0.7	0.98	110
4	1.0	1.40	157
5	1.2	1.68	168

5.2. Annualised risk

The annual average rate of overturning-induced collapse, λ_C, can be computed using the integral of Equation (7):

$$\lambda_C = \nu_{min} \int_0^{z_{max}} P(C|z) f(z) dz \tag{7}$$

where $P(C|z)$ is the collapse fragility function as defined by Equation (1), and $f(z)$ is the probability density function (PDF) of the hazard with ν_{min} being the annual average rate of exceeding the minimum threshold hazard level z_{min} that is considered in the calculation. The integration is performed from $z = z_{min}$ (or 0 for simplicity) up to a maximum limit z_{max}, which is the upper bound earthquake ground motions to be discussed below. The relationship between the hazard function, $H(z)$, for the ground motion parameter z, and the PDF of the hazard is expressed in Equation (8).

$$f(z) = \frac{d}{dz} F(z) = \frac{d}{dz}\left[1 - \frac{H(z)}{\nu_{min}}\right] \tag{8}$$

where $F(z)$ is the cumulative density function (CDF) of the hazard and its first derivative is the PDF. Hence, the probability of overturning-induced collapse in 1 year, $P_a(C)$, can be approximated by Equations (9) and (10).

$$P_a(C) \approx \lambda_C = \int_0^{z_{max}} P(C|z) \left|\frac{dH(z)}{dz}\right| dz \tag{9}$$

$$P_{50}(C) = 1 - e^{-50\lambda_c} \approx 50\lambda_C \tag{10}$$

The hazard parameter, z, is represented by *PDD* of the precast column in this study. $|dH(z)/dz|$ is the absolute value of the first derivative of the hazard function $H(z)$.

The annual average rate of collapse has then been computed for all the combinations of the four column sizes and the five soil sites, under unidirectional and bidirectional earthquake actions, respectively. In the present study, the upper limit of hazard is chosen as the median prediction of spectral acceleration at T = 1.5 s, i.e. $S_a(1.5)$, that can be generated by an Mw 7.5 thrust faulting earthquake occurring at a close distance of 3 km on an unidentified fault (a scenario assumed in Somerville et al. 2013), which is equal to 0.52 g on rock sites and has an annual frequency of exceedance around 3×10^{-6}. This means that the hazard function of Equation (3) has to be extrapolated to higher return periods for the integration, which represents an additional element of uncertainties in the computed collapse risk. A sensitivity study shows that the total collapse probability estimated for any of the 20 building-site combinations can be elevated by around 2.5×10^{-6} if ground motions of lower frequencies of exceedance are included in the computation, which is considered negligible in comparison with the target collapse risk limit that is in the order of 10^{-4}.

Figure 4(a,b) show the unidirectional collapse risk in 1 year and that over a notional design life of 50 years, respectively, for all 20 building-site combinations. Generally, smaller columns have a higher risk of failure as they are more prone to overturning under earthquake ground shakings. As the *PDDs* on more flexible soil sites (with site natural period of 1.0 s and 1.2 s) are higher, the chances of failure are also higher for those cases. All of them are lower than the annual risk limit of 2×10^{-4} or the limit of 1% in 50 years stipulated in IBC (2012) and ASCE/SEI 7–10. The highest probability of collapse amongst all 20 building-site combinations is 1.6×10^{-4} in a year, which corresponds to a 0.8% chance of collapse in 50 years.

Another benchmark that can be adopted for comparison is the collapse risk limit of 0.25% in 50 years (i.e. a quarter of that stipulated in IBC (2012) and ASCE/SEI 7–10) recommended for Europe (Silva, Crowley, and Bazzurro 2016). This is very close to the proposed risk limit of 0.3% in 50 years for this type of RC structure (Tsang and Wenzel 2016), which was derived for controlling the individual annual fatality risk in an ordinary building to the tolerable level of 10^{-6} (ISO 2394 1998). It is shown that a column of dimension smaller than 400 mm that is sitting on a flexible soil site with site natural period exceeding 0.6 s may have a higher risk than the limit. The general risk limit of 0.5% proposed by Tsang and Wenzel (2016) for all types of structures is also superimposed in Figure 4.

When bidirectional earthquake actions induced on the columns are significant, a factor of 0.625 has to be applied to the failure drift capacity, which is calculated based on the ratio of 0.6 between the non-concurrent maximum displacement in the smaller loading direction and the larger loading direction. With the adjusted $\overline{PDD}_b$ obtained from Equation (2), the bidirectional collapse probability of the soft-storey buildings can be computed as shown in Figure 5(a,b). Clearly, the risk becomes higher than the annual risk limit of 2×10^{-4} or 1% in 50 years which is stipulated in IBC (2012) and ASCE/SEI 7–10, when the building is sitting on a Class D or E site with column size smaller than 400 mm.

5.3. Risk deaggregation

Deaggregation of the collapse risk can be performed in order to examine the relative contribution of the risk from ground motions of different return periods. This analysis can indicate the range of return periods that controls the total collapse risk, which would have important implications for seismic design and analysis. This also provides a tool to check whether the

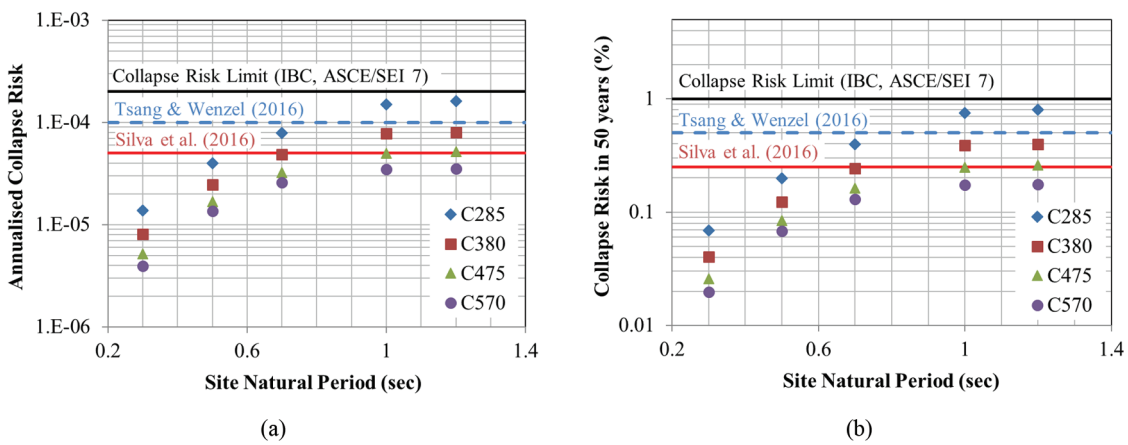


Figure 4. Collapse risk of soft-storey buildings with various column sizes located on different soil sites in Melbourne, Australia under unidirectional earthquake actions: (a) annualised, (b) over a notional design life of 50 years.

computed risk values have already reached the saturation level at the upper limit of hazard in the integration.

The cumulative risk from zero up to the value of z, defined as $P(C,z)$, for 1 year and 50 years can be computed from Equations (11) and (12) respectively. On the other hand, the contribution of risk from ground motion level z, defined as a fraction of the probability $p(C,z)$ or an incremental change of the exceedance rate $\Delta\lambda_{C,z}$ (i.e. numerically from $z-0.005$ to $z+0.005$) (in the unit of g), for 1 year and 50 years can be computed from Equations (13) and (14) respectively.

$$P_a(C,z) \approx \lambda_{C,z} = \int_0^z P(C|z)\left|\frac{dH(z)}{dz}\right|dz \tag{11}$$

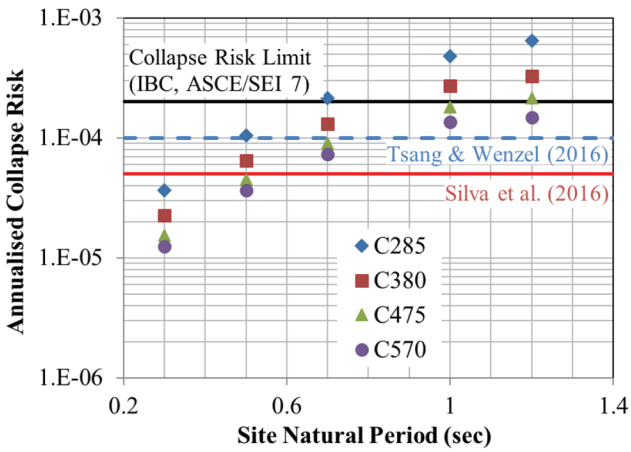
$$P_{50}(C,z) = 1 - e^{-50\lambda_{c,z}} \approx 50\lambda_{C,z} \tag{12}$$

$$p_a(C,z) \approx \Delta\lambda_{C,z} = \int_{z-0.005}^{z+0.005} P(C|z)\left|\frac{dH(z)}{dz}\right|dz \tag{13}$$

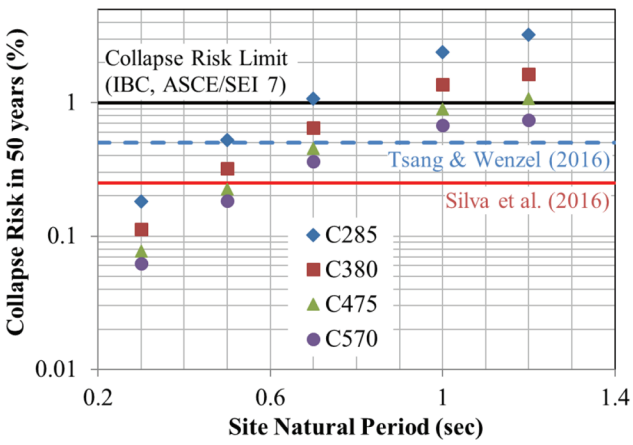
$$p_{50}(C,z) = 1 - e^{-50\Delta\lambda_{C,z}} \approx 50\Delta\lambda_{C,z} \tag{14}$$

Figure 6(a,b) show the cumulative collapse risk in 50 years of the soft-storey buildings with columns of the smallest dimension (i.e. 285 mm) located on five different soil sites subjected to (a) unidirectional and (b) bidirectional earthquake actions, respectively, based on Equations (11) and (12). Instead of plotting the cumulative probability against the ground motion level, z, the corresponding return period is shown. It is observed that the cumulative probability starts to saturate at return period around 2×10^5. This means that the results presented in Section 5.2 are not sensitive to the choice of maximum limit, z_{max}, in the integration using Equation (7). Also, the hazard data for annual rates down to 2×10^{-5} presented in Somerville et al. (2013) can be considered sufficient, as extrapolation was required for a limited range only.

The contribution of collapse probability from ground motions of different return periods can be calculated using Equations (13) and (14) and the results are shown in Figure 7(a,b). If the horizontal axis is the ground motion level, z, the plot would become



(a)

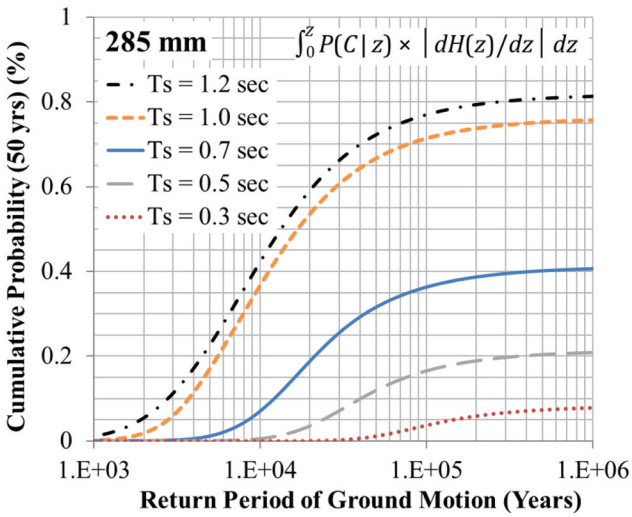


(b)

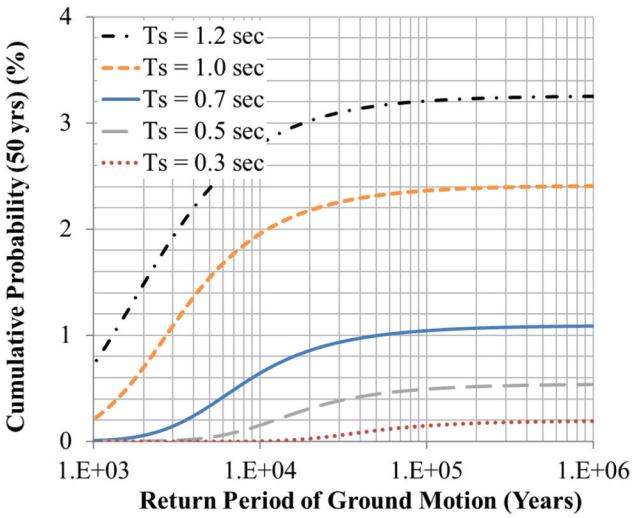
Figure 5. Collapse risk of soft-storey buildings with various column sizes located on different soil sites in Melbourne, Australia under bidirectional earthquake actions: (a) annualised, (b) over a notional design life of 50 years.

a probability distribution function (PDF), whereas the total area under each curve is not a unity but the total collapse probability for the respective column. It is clearly seen that the range of return periods that contributes most to the total collapse risk shifts to the lower end for higher risk scenarios. For more flexible sites (i.e. with higher site natural periods) under unidirectional earthquake actions (Figure 7(a)), ground motions with return periods between 2,000 years to 20,000 years control the total collapse risk on Class E sites (site natural period higher than 0.9 s), whilst it becomes 10,000 years to 100,000 years for Class C sites (site natural period lower than 0.6 s). Taking the case with bidirectional actions (Figure 7(b)), ground motions with return periods between 500 years to 5,000 years contribute significantly to the risk on Class E sites, whilst it would be in the order of 10,000 years for more typical sites (Class C and D).

It is noteworthy that the controlling ranges of return periods are well beyond the design life safety (LS) or ultimate limit state (ULS) level that is based on a design return period of 500 years in Australia as well as many other countries. It indicates the importance of understanding the seismic performance of buildings at higher return period levels and the need to evaluate the residual risk of buildings. Collapse prevention (CP) or near collapse (NC) limit state should be introduced as part of a performance-based seismic design framework. When selecting ground motions for conducting collapse assessment through dynamic analysis, earthquake scenarios corresponding to a return period in the order of



(a)



(b)

Figure 6. Cumulative collapse risk (in %) in 50 years of soft-storey buildings with columns of the smallest dimension (i.e. 285 mm) located on five different soil sites under (a) unidirectional and (b) bidirectional earthquake actions respectively.

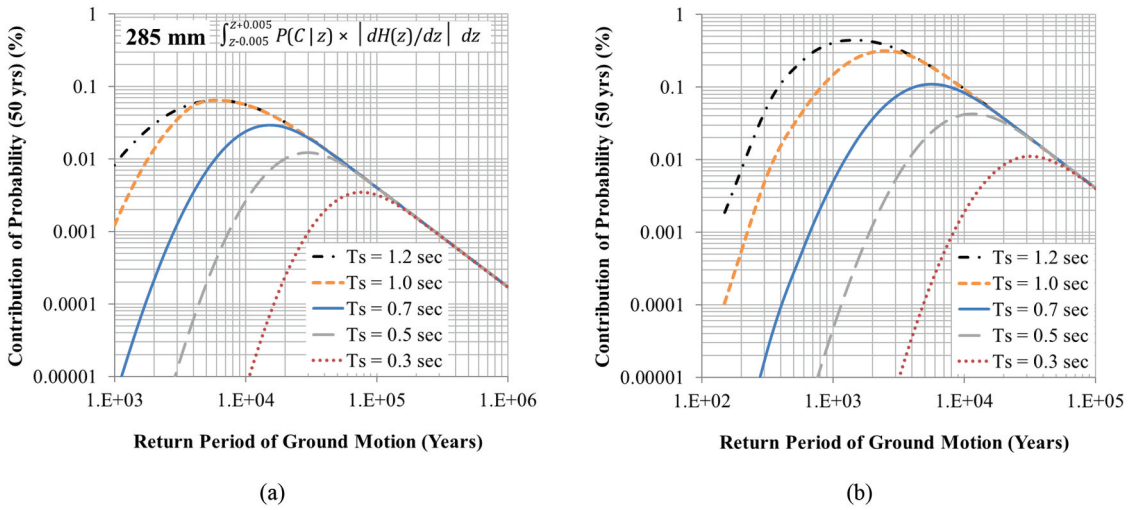


Figure 7. Deaggregation of the collapse probability (in %) in 50 years of soft-storey buildings with columns of the smallest dimension (i.e. 285 mm) located on five different soil sites under (a) unidirectional and (b) bidirectional earthquake actions respectively.

10,000 years would be more appropriate, especially in regions of lower seismicity.

6. Conclusions

This paper presents the analysis results of the collapse risk of soft-storey buildings supported by precast RC columns under unidirectional and bidirectional earthquake actions, respectively. Columns of four different sizes and five site conditions have been considered (i.e. 20 building-site combinations). The collapse fragility functions of the precast columns were developed based on results from full-scale field testings and supplemented by laboratory and analytical investigations. A ground motion recurrence relationship for rock sites reported in the literature has been adopted alongside a theoretical model for estimating resonant-like nonlinear amplification behaviour in different soil sites that are classified based on site natural period.

The results have led to the following useful implications for seismic design and analysis, which will become more important when risk-based design requirements are introduced. It is found that typical soft-storey buildings under unidirectional earthquake actions are deemed safe if the computed risk values were benchmarked against the risk-based requirements in IBC (2012) and ASCE/SEI 7–10. Columns with dimensions smaller than 400 mm that are sitting on a flexible soil site with site natural period exceeding 0.6 s are found to be more vulnerable and have a higher risk than the limits recommended recently by various international groups of researchers. The collapse probability is significantly lower if the structures (1) have larger column size, (2) located on stiffer sites with site natural period lower than 0.6 s, and (3) are confined to unidirectional actions by introducing lateral load resisting elements along one axis.

The range of return periods of ground motions that controls the collapse risk is in the order of 10^4, which is well beyond the life safety (LS) or ultimate limit state (ULS) level in existing design. Collapse prevention (CP) or near collapse (NC) limit state has to be introduced as part of a performance-based seismic design framework. A better understanding of the seismic performance of buildings at higher return period levels is important for controlling the residual risk of collapse and casualties to a tolerable level.

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