Review of Methodologies for Seismic Vulnerability Assessment of Buildings

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ABSTRACT:

Earthquake action has only been considered in structural design in Australia since the early 1990s. With a very low building replacement rate many Australian buildings are vulnerable to major earthquakes and pose significant risk to lives, properties and economic activities. The vulnerability of buildings was evident in the Newcastle Earthquake of 1989 which has been reported to have caused damage to more than 70,000 properties and an estimated total economic loss of AU$ 4 billion.

Models that are capable of predicting potential economic loss in future earthquakes are fundamental in the formulation of risk mitigation and retrofitting strategies. An important aspect of the damage loss modelling is an accurate and reliable assessment of the seismic vulnerability of buildings. This paper presents a review of the existing techniques and methodologies that have been developed for the assessment of seismic vulnerability of buildings. Key components of the methodologies including selection of intensity measures, classification of building types, definition of building parameters, selection of analysis method and definition of damage states will be discussed. The applicability of the existing methodologies especially the classification of building types to Australia forms of constructions will be evaluated.

Keywords: vulnerability assessment, economic loss, earthquakes, reinforced concrete, unreinforced masonry

1. Introduction

Earthquake damage loss modelling has gained popularity driven by experiences from past events (for example, the Northridge and Kobe earthquakes which have caused an estimated economic loss of $44 billion and $100 billion, respectively) and the needs of end users such as emergency planners, government bodies and insurance industry. A large number of methods have been developed for estimation of earthquake losses (e.g. Blume et al., 1977; Insurance Services Office, 1983; ATC, 1985; ATC, 1997; National Research Council, 1999). Earthquake loss estimation software tools have been developed, with HAZUS multi-hazard software (FEMA, 2012) being the most notable example. Other earthquake loss estimation software tools have been developed for other regions around the world, for example, SELENA (SEimic Loss EstimatioN using a logic tree Approach) (Molina and Lindholm, 2005; Molina et al., 2010) and DBELA (Displacement-Based Earthquake Loss Assessment) for Europe (Crowley et al., 2006) and EQRM (Geoscience Australia’s Earthquake Risk Model) for Australia (Robinson et al., 2006).

Central to an earthquake loss model is the assessment of vulnerability of structures. The seismic vulnerability of a structure can be described as the susceptibility to damage under a given intensity of earthquake shaking. The aim of the assessment is to obtain the probability of a certain level of damage to a given building class to be exceeded for a given scenario of earthquake. Numerous assessment techniques have been proposed over the past decades. The assessment techniques can be broadly divided into two categories: empirical methods which are based on observation of damages and analytical methods which rely on assessing structural performance through analytical procedures.

There are important aspects associated with the derivation of vulnerable functions through analytical methods. These aspects include selection of parameters for measurements of intensity, selection of samples of buildings which are representative of a building class, selection of analysis method and structural models, selection of representative earthquakes, classification of buildings and definition of damage states. This paper presents a review of existing methodologies for seismic vulnerability assessments in the contexts of these aspects. It is noted that only aspects associated with the process of obtaining vulnerability functions, as opposed to aspects associated with the whole process of seismic vulnerability assessment for a region, are discussed in this paper. A critical step in conducting seismic vulnerability assessment for a region is seismic hazard analysis which involves identification of potential seismic sources for the region, magnitude recurrence modelling, ground motion predictions, integration of contributions from multiple sources and analysis of site conditions. Seismic hazard analysis is an important study in its own right and is outside the scope of this paper. The applicability of the methodologies especially on the classification of buildings to Australian forms of constructions is also discussed.

2. Measures of intensity

Techniques for seismic vulnerability assessment of buildings have been developed since the early 70's. Macroseismic intensity has traditionally been adopted as the intensity parameter at which damages are being measured (e.g. Whitman et al., 1973; Braga et al., 1982; Di Pasquale et al., 2005). Macroseismic intensity is not a continuous parameter and consequently the probability of damage was often presented in a discrete form.

Spence et al. (1992) introduced the Parameterless Scale Intensity (PSI) which allows continuous vulnerability functions to be derived for various types of buildings. Sabetta et al. (1998) used post-earthquake surveys to derive vulnerability functions based on ground motion parameters including Peak Ground Acceleration (PGA), Effective Peak Acceleration (EPA which is defined as the maximum acceleration between natural period of 0.1 to 0.5 s) and Arias Intensity (AI which is defined as the integral of the square of the acceleration time history). PGA has also been adopted as an intensity measure for vulnerable functions in recent studies (e.g. Decanini et al., 2004).

Empirical and analytical vulnerability functions have also been developed based on spectral acceleration or spectral displacement. The development was motivated by the fact that PGA cannot represent the frequency content of the ground motions. Rosetto and Elnashai (2003; 2005) adopted the 5% damped spectral displacement value at the elastic fundamental period as intensity measures and demonstrated that such parameter correlates better with the damage level than PGA. Singhal and Kiremidjian (1996) used the average spectral acceleration values over various period ranges as intensity measures. Colombi et al. (2008) used the inelastic displacement value based on the *Substitute Structure* approach (Shibata and Sozen, 1976) and the predicted elastic displacement value which was estimated by ground motion prediction equations developed by Faciolli et al. (2007).

3. Selection of representative building samples

The selection of building samples that will represent a class of buildings is an important step in analytical seismic vulnerability assessment. Important parameters characterising seismic capacity and response include material properties (strength of material), building dimensions (total height/storey height, number of storeys, plan dimensions), structural detailing and geometric configuration (D'Ayala et al., 2014).

Due to the computationally intensive nature of analytical seismic vulnerability assessment, existing studies often only consider variation in material properties in their selection of building samples. For example, Singhal and Kiremidjian (1996) derived analytical vulnerability functions for reinforced concrete frames by considering variation in steel yield strength and concrete compressive strength. Similarly, Shinozuka et al. (2000) used variation in compressive strength of concrete and yield strength of steel to construct vulnerability curves for bridges. Masi (2003) derived vulnerability functions for different types of reinforced concrete frames (bare, regularly infilled and pilotis), however considered only variation in reinforcement contents in some of the structural members.

4. Choice of analysis methods and selection of ground motions

Another important step in the derivation of analytical vulnerability functions is the choice of analysis method for evaluating the median and probability distribution of structural responses (i.e. demand) of buildings. Nonlinear dynamic analysis has been adopted by numerous researchers as it is viewed to be able to represent the actual effects of ground motion characteristics (e.g. Singhal and Kiremidjian, 1996; Mosalam et al., 1997; Masi, 2003; Kwon and Elnashai, 2006).

Guidelines for Analytical Vulnerability Assessment has recently been produced in the context of the Vulnerability Global Component project of the Global Earthquake Model (GEM). In the guidelines (D'Ayala et al., 2014), three analysis options with decreasing order of complexity have been provided: incremental dynamic analysis which is based on nonlinear time history analyses using ground motion inputs that are incremented until either global dynamic or numerical instability occurs (as indicated by the point where the curves in Figure 1 become flat); nonlinear static analysis which relies on the capacity curves obtained from pushover analyses; and nonlinear static analysis that is based on a simplified mechanism model (as opposed to pushover analyses).

Nonlinear dynamic analyses are generally computationally intensive especially if numerous analyses are required to represent a building population and ground motion uncertainties. As a result, compromises in regards to the structural modelling were often made. Single-degree of freedom idealisation has often been made (e.g. Mosalam et al., 1997). If a multi-degree of freedom model is adopted, the model normally assumes that buildings are regular in plan and height (e.g. Singhal and Kiremidjian, 1996; Rosetto and Elnashai, 2005).

In an attempt to minimise computational efforts, methods based on nonlinear static procedure have often been adopted. For example, Rosetto and Elnashai (2005) and Uma et al. (2014) adopted the capacity spectrum method, which is based on comparison between an inelastic response spectrum and a capacity curve (Figure 2). Inelastic earthquake spectra can be derived from ground motion time histories which can match with the target spectrum (as adopted by Rosetto and Elnashai (2005)), or ground motion records of an event (as adopted by Uma et al. (2014)) or ground motion prediction equations (as adopted by earthquake loss models such as HAZUS (FEMA, 2012) and EQRM (Robinson et al., 2006)).

When nonlinear dynamic procedures are adopted, selection of input motions is crucial. The selection of input motions contributes to the uncertainties in vulnerability analyses but there are currently no consistent guidelines on the selection of ground motions for vulnerability assessments. Seismic design codes and assessment guidelines such as Eurocode 8 (CEN, 2004) and FEMA-356 (ASCE, 2000) require the use of a minimum of three sets (2 horizontal components or 3 components) of ground motions for time history analysis. The number of ground motion sets required is dependent on factors such as whether mean values or distribution of responses are required, the expected degree of inelastic response and the number of modes contributing significantly to the response quantities. The National Institute of Standards and Technology (NIST) (NEHRP, 2011) recommends taking peak as opposed to mean maximum responses where only three sets of ground motions are used. Seven sets of ground motions are required to obtain the average maximum responses whilst no less than 30 sets of motions are required for construction of vulnerability curves. 11 sets of ground motions are required to perform incremental dynamic analysis (ATC, 2009; ATC, 2012; D'Ayala, 2014).

Most guidelines require earthquake ground motion amplitudes to be scaled in order to match the target spectrum over a certain period range. Further, the selected records should have magnitudes, fault distances and source mechanisms that are representative of the earthquake scenarios that control the target spectrum (CEN, 2004; ASCE, 2000; ATC, 2012).

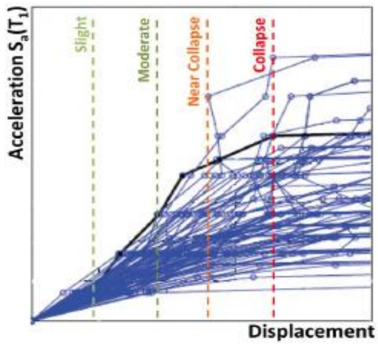


Figure 1 Example plot of incremental dynamic analysis curves (D'Ayala et al., 2014)

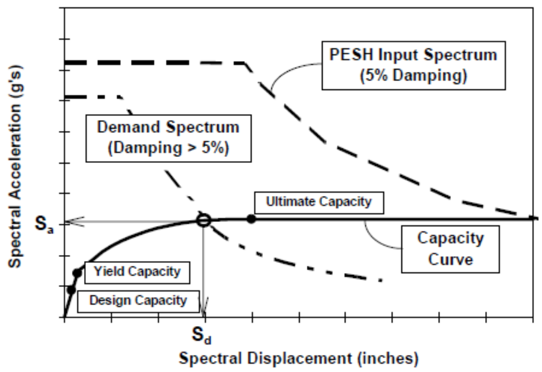


Figure 2 Capacity spectrum method (FEMA, 2012)

5. Classification of buildings and definition of damage levels

Buildings have traditionally been classified in accordance with their construction materials as they are considered to be directly related to the seismic vulnerability of the buildings. Braga et al. (1982) classified buildings into three vulnerability classes (A, B and C) which are directly related to three types of structures with different construction materials: buildings made of fieldstone (type A), bricks (type B), or concrete frame structures (type C). The classification of buildings by Braga et al. (1982) has been expanded by Spence et al. (1992) to include more information of construction materials and building types. An additional vulnerability class D has been included by Dolce et al. (2003) to account for reinforced concrete frames or walls constructions with moderate level of seismic design. Two additional vulnerability classifications have been added in the European Macroseismic Scale 1998 (EMS-98) (Grünthal, 1998) to incorporate steel and timber constructions. Earthquake risk models such as HAZUS and EQRM classify buildings according to types of construction materials, lateral load resisting elements and heights of buildings (FEMA, 2012; Robinson et al., 2006). Two types of lateral load resisting elements have generally been included: moment resisting frames and shear walls. Concrete frames featuring soft-storeys which are typical construction forms in Australia have been incorporated in EQRM (Robinson et al., 2006).

6. Definition of damage levels

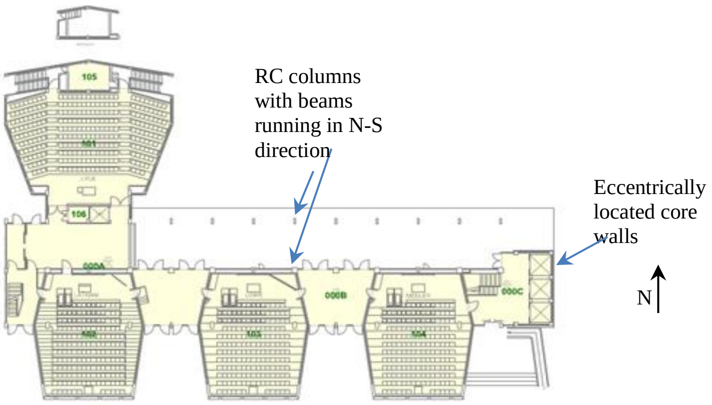
The definition of damage levels for the assessment of buildings is also important in the construction of vulnerability curves. For empirical vulnerability functions, damage levels of buildings are generally characterised using descriptive damage states. For example, EMS-98 defines five levels of damage states and provides qualitative descriptions for each level. The damage levels have been adopted in various empirical vulnerability functions (e.g. Dolce et al., 2003; Decanini et al., 2004). In the earlier assessments, the probability of damage was represented by the damage probability matrices (DPMs) which present the buildings vulnerability in a discrete form. The use of DPMs for the probabilistic prediction of damage was first proposed by Whitman et al. (1973) and since adopted by other seismic vulnerability assessments worldwide (e.g. Braga et al., 1982; Dolce et al., 2003).

For analytical vulnerability functions, different levels of damage are commonly defined based on drifts that have been calibrated to observations of building damages or experimental results (e.g. Singhal and Kiremidjian, 1996; Rossetto and Elnashai, 2005). GEM guidelines (D'Ayala et al., 2014) define four structural damage states: Slight (defined as the limit of elastic behaviour), Moderate (corresponds to the peak lateral load bearing capacity), Near Collapse (corresponds to the maximum controlled deformation) and Collapse. The definition of each damage state and the corresponding inter-storey drift value can be obtained from various seismic assessment guidelines such as ATC58-2 (ATC, 2003), Eurocode 8 (CEN, 2004) and FEMA-356 (ASCE, 2000). The probability of damage in analytical vulnerability assessment is normally presented in a continuous form.

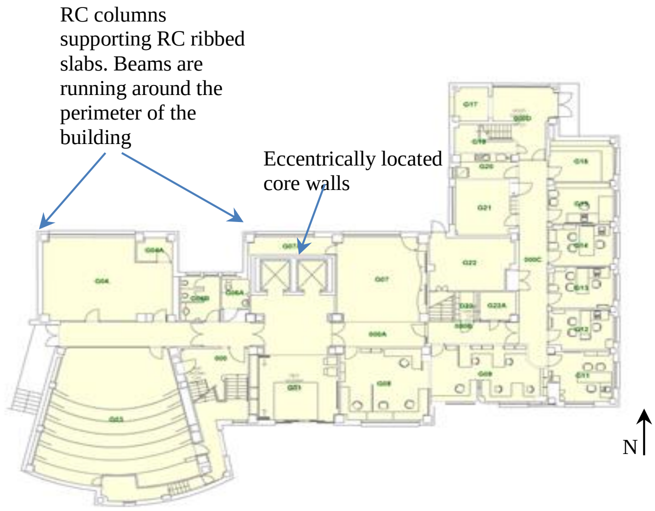
7. Applicability of the existing methodologies in the Australian context

Australian existing building stock mainly consists of unreinforced masonry and reinforced concrete buildings. Reinforced concrete buildings are commonly supported laterally by shear/core walls or combined shear/core walls and frames (as shown by examples of layouts presented in Figure 3), but are rarely supported by moment resisting frames alone. These lateral resisting elements are normally designed with low level of ductile detailing. Unreinforced masonry walls and nonductile reinforced concrete frames have been identified to be vulnerable in earthquake excitations, as demonstrated by structural damages observed after the 1989 Newcastle earthquake event (IEAust, 1990).

The deformation behaviour of moment resisting frames under earthquake excitations is different to that of shear/core walls as demonstrated by results of parametric studies performed by Fardipour et al. (2011). Importantly, the deformation behaviour of walls within buildings was generally found to dominate the deformation behaviour of the buildings (Figure 4). It should also be noted that the location of core/shear walls is often highly eccentric (as indicated in Figure 3) which could result in amplification of the displacement imposed on the reinforced concrete frames located at the perimeter of the buildings. In accordance with the existing guidelines for seismic vulnerability assessments, reinforced concrete frames have often been assessed in isolation from the overall structures as they are considered as primary lateral load resisting elements. In view of the types of construction forms in Australia and how the deformation behaviour is controlled by the types of lateral load resisting elements, it is important to consider the behaviour of structural systems as a whole in the seismic vulnerability assessments of reinforced concrete buildings.



(a) Building A



(b) Building B

Figure 3 Examples of building layout

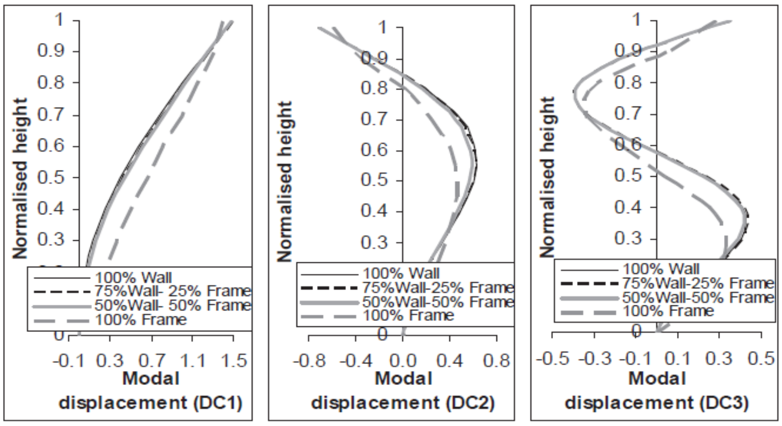


Figure 4 Displacement behaviour of frames and walls (from Fardipour et al., 2011)

Many existing buildings are supported by reinforced concrete frames with masonry infills. The buildings also often feature an open plan on the ground floor that causes an abrupt change in the lateral stiffness along the height of the buildings. The effects of soft-storey features on the displacement behaviour of buildings are demonstrated in Figure 5 based on elastic modal dynamic analyses conducted by Sofi et al. (2013).

Response spectrum analyses have been conducted based on the modal displacements presented in Figure 5. Accelerograms used in the analyses were generated using program GENQKE (Lam et al., 2000) based on an earthquake scenario that produces peak ground velocity on rock of 60 mm/sec. The program SHAKE (Ordonez, 2013) has been used to generate accelerograms that are representative of earthquake excitations on class C and D sites in accordance with AS1170.4-2007 (SA, 2007). The displacement and inter-storey drift profiles of buildings with and without a soft-storey are compared in Figure 6. It is demonstrated that the building featuring a soft-storey is subject to a larger displacement and inter-storey drift on the ground floor under the earthquake excitation than the building without a soft-storey. The larger displacement and inter-storey drift could cause concentration of damages on columns located at the ground floor. In view of the displacement behaviours and types of damages that could occur on the buildings, reinforced concrete frames featuring soft-storeys should be incorporated in the classification of buildings for seismic vulnerability assessments.

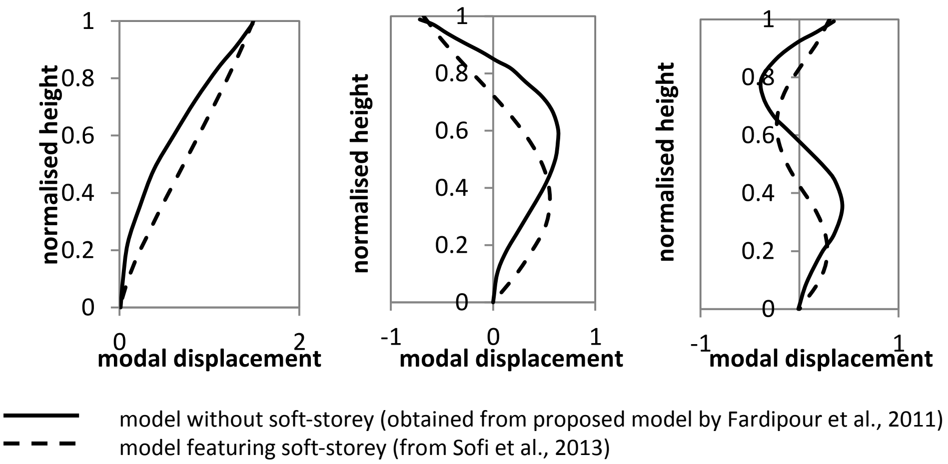


Figure 5 Comparison of modal displacements of buildings with and without soft-storey

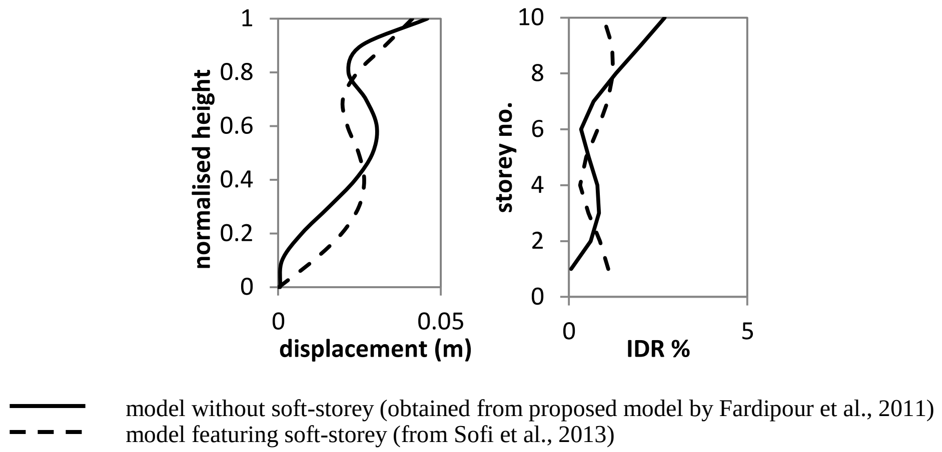
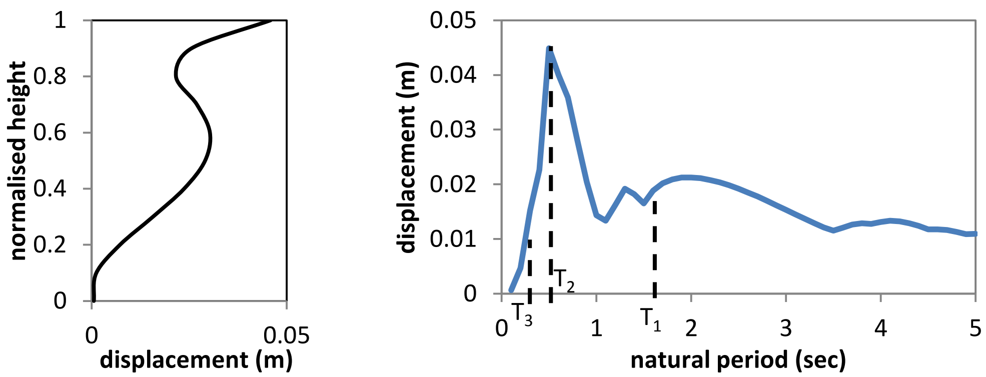


Figure 6 Comparison of displacement and inter-storey drift profiles of buildings with and without soft-storey

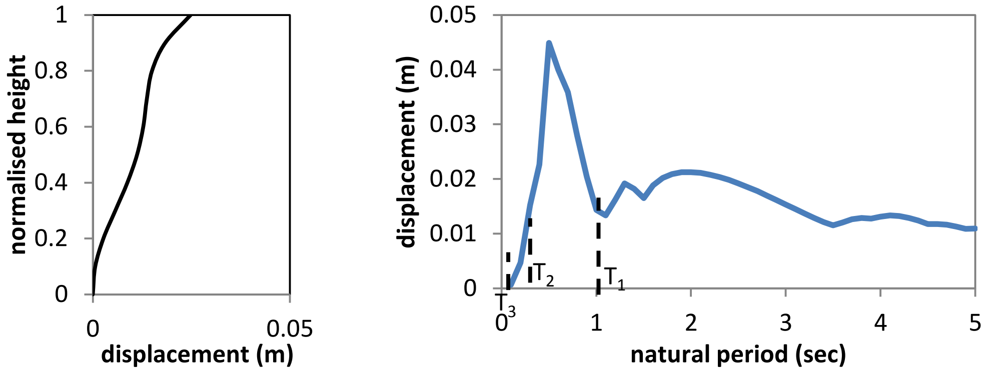
The height of buildings has also been incorporated in the classification of buildings in seismic vulnerability assessments (Section 5). The buildings are generally classified into low rise (up to 3 storeys), medium rise (between 3 to 8 storeys) and high rise (8 storeys and above). To minimise computation time, seismic vulnerability assessments have normally been based on single-degree of freedom idealisation assuming first mode of response (as discussed in Section 4).

Buildings with first modal period of 1.6 and 1.0 sec representing approximately 20-storey and 10-storey buildings, respectively, were subject to input motions representing earthquake excitations on class C and D sites. Response spectrum analyses were performed based on modal displacements presented in Figure 5. The 2nd and 3rd modal periods of the buildings are assumed to be 0.3 and 0.1 of the first modal period, respectively, based on the parametric studies performed by Fardipour et al. (2011). Figures 7a and 8a present the displacement profiles of the buildings subject to class C site input motions. The displacement response spectrum is plotted in Figures 7b and 8b along with the first three modal periods of vibration of the buildings. Results from the analyses indicate that the displacement behaviour of both buildings is affected by the higher modes. Hence the idealisation into single-degree of freedom adopted in the analysis and modelling options needs to incorporate these effects. Importantly, the effects of higher modes were found to be far more pronounced in the 20-storey buildings (compare Figure 7a and 8a). This behaviour can be explained by the high response spectral displacement value at the second modal period of vibration T_2 of the 20-storey building as the period coincides with the dominant site period.



(a) displacement profile (b) displacement response spectrum

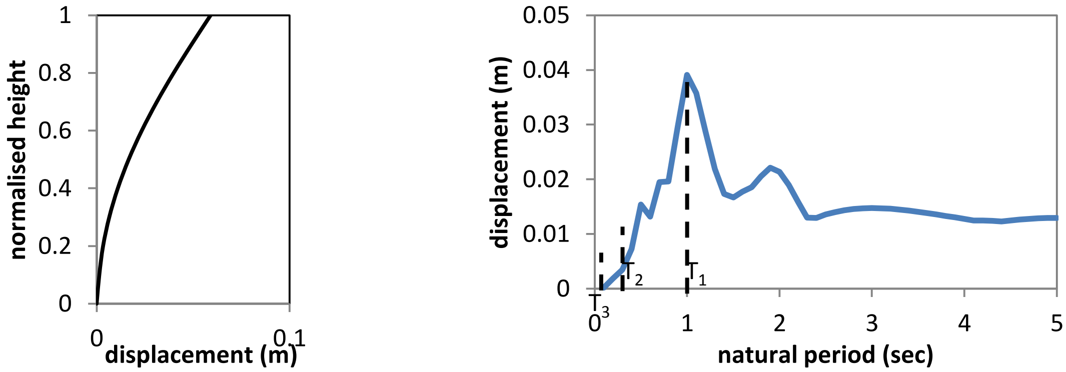
Figure 7 20-storey building (T_1 = 1.6 sec) subject to Class C input motions



(a) displacement profile (b) displacement response spectrum

Figure 8 10-storey building (T_1 = 1.0 sec) subject to Class C input motion

Results from response spectrum analyses of the buildings subject to class D site earthquake excitations are presented for comparison in Figure 9. The figure demonstrates that the higher mode effects are reduced as a result of the modal periods of vibration being on the inclining part of the displacement response spectrum (as shown in Figure 9b). This may well be the case in regions of high seismicity where the inclining part of the displacement response spectrum (the velocity controlled region of the response spectrum) of earthquakes in these regions span over a long period range. The classification of buildings into three types (low, mid and high-rise) according to the height of the buildings may be appropriate for these regions. However, the comparative analyses have demonstrated that in regions of low to moderate seismicity such as Australia the deformation behaviour of buildings taller than 20-storeys may be categorically different to that of 10-storey buildings and should be considered in seismic vulnerability assessments in these regions.



(a) displacement profiles (b) displacement response spectrum

Figure 9 10-storey building (T_1 = 1.0 sec) subject to Class D input motion

8. Concluding remarks

Existing methodologies for seismic vulnerability assessments have been reviewed in the context of the selection of parameters for measurement of vulnerability and intensity, the selection of samples of building which are representative of building class, the selection of analyses and structural models and representative earthquakes, the classification of buildings and definition of damage states. Spectral displacement and spectral acceleration values were generally viewed as better parameters in representing the intensity of earthquakes than PGA and intensity parameters due the ability of the spectral values to better represent the frequency characteristics of the earthquakes. Although non-linear time history analyses have been generally viewed to better represent the effects of ground motion characteristics on the response of structures, they are considered to be computationally intensive. As a result, compromises were often made in the assessments such as idealisation of structural models into single-degree of freedom systems and two-dimensional models ignoring the effects of asymmetry.

The classification of buildings in the existing methodologies for seismic vulnerability assessments has been reviewed and its applicability to the Australian construction forms has been discussed. Reinforced concrete frames have been identified as one of the most vulnerable construction forms in Australia. However, the reinforced concrete frames are seldom used as primary lateral load resisting elements in the buildings. The buildings were generally found to be laterally supported by core/shear walls that are highly eccentric. Hence the seismic vulnerability assessments of reinforced concrete frames have to incorporate the behaviour of structural systems as a whole as opposed to the behaviour of reinforced concrete frames in isolation. Many Australian buildings features an open plan on the ground floor and this has been identified to cause larger inter-storey drifts which could cause concentration of damages on columns located on the ground floor. It was also demonstrated that the deformation of 20-storey buildings or higher could be categorically different to that of 10-storey buildings. Hence the current definition of high-rise buildings in the existing methodologies (8-storeys and higher) may be too broad.

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