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Generation of analytical fragility curves for Ghanaian non-ductile reinforced concrete frame buildings

Mark Adom-Asamoah

Department of Civil Engineering, College of Engineering, Kwame Nkrumah University of Science and Technology, Kumasi, Ghana. E-mail: madom-asamoah.coe@knust.edu.gh.

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This study is on seismic safety evaluation of non-ductile reinforced concrete (RC) frame buildings in Ghana. Generic 3-, 4- and 6-storey non-ductile RC buildings were characterised for assessment. The generic buildings were designed according to British Standard (BS) 8110. The fragility curve parameters were generated using inelastic time history analyses for seismic demand and inelastic pushover analyses for structural capacity of the buildings. As a result of lack of real time histories in Ghana, 3 suites of 100 synthetic time history records each were generated in order to generate lower and upper bounds for inelastic dynamic response. Parameters of collapse fragility curves with 50% probability of exceedance were established for the generic non-ductile Ghanaian frame buildings using short duration near-fault like records. All the curves represent the probability of exceeding the collapse limit state as a function of the peak ground acceleration (PGA) and spectral acceleration at the fundamental mode period of the generic buildings. Using the data from the nonlinear dynamic analysis of the generic buildings under all 300 synthetic time histories, a pair of three sets of fragility curves was developed. Results obtained showed that the generic non-ductile 3 to 6 storey RC frame buildings subjected to near-fault like ground motions may record high probabilities of collapse, if they are situated in 0.25 to 0.35 g PGA seismic zones.

Key words: Earthquake, seismic vulnerability, fragility curve, damage index, reinforced concrete.

INTRODUCTION

Non-ductile reinforced concrete (RC) frame buildings are designed to resist gravity loads and lateral wind loads. In most low to medium storey buildings (typically 1 to 7 storeys), the wind load may be negligible and the design is based on the gravity and nominal lateral load. Experimental and analytical studies by researchers (Bracci et al., 1992a, b, c; Balendra et al., 1999) have shown that non-ductile RC frames designed only to resist gravity load have limited lateral strength that contribute to resist minor to moderate earthquakes. The estimated number of vulnerable RC concrete frame buildings in developing countries is very high. In areas of medium-to-large, but infrequent events, such as, the India, Iran, Pakistan, Haiti and Ghana, seismic detailing of buildings are uncommon. The behaviour of non-ductile RC frames is of special interest because of their susceptibility to softening and sudden collapse in major earthquakes. Recent earthquakes have proved that thousands of lives have been lost due to the collapse of non-ductile buildings in low-medium seismic risk zones of the world. Research activity on seismic vulnerability of non-ductile buildings is on-going in areas like Central United States, Eastern Canada and Australia. Efforts to quantify the seismic performance of non-ductile RC frame buildings in Ghana by analytical or experimental methods are inadequate (Adom-Asamoah and Taylor, 2006).

To address this need, this study seeks (i) to estimate the levels of seismic loads that generic 3-, 4- and 6-storey moment resisting non-ductile RC frames can resist before collapse, and (ii) to establish simple fragility curves that can be used as a field document in the vulnerability assessment of such population of buildings in a low - medium seismic areas such as Ghana.

Selection of generic RC frames

The capital of Ghana, Accra, which lies within the low seismic region of West Africa, is the most seismically active area in the region according to recent instrumental recordings and historical records. A review of the historical seismicity of the West African sub-region (Ambraseys and Adams, 1986) has shown that Accra was partially destroyed during three major earthquakes in 1862, 1906 and 1939.

The building stocks in Accra-Ghana (about 95% of which are reinforced concrete frame buildings) are characterized by poor quality of building materials and rampant building planning practices. Some of the typical deficiencies observed in non-ductile reinforced concrete buildings in Ghana include (a) inadequate transverse reinforcement in the critical regions, (b) insufficient lap splice and anchorage lengths, (c) no consideration of interaction between columns and infill walls, (d) discontinuous longitudinal reinforcement and (e) inadequate foundation design. These deficiencies result in large drifts under low-to-moderate shaking, hinging in the columns and extensive damage to non-structural elements and contents. In order to obtain meaningful results for practical use, 3 generic non-ductile RC frame buildings; a 3-storey 3-bay, a 4-storey 2-bay and a 6-storey 3 bay buildings were selected for this study. The selection of these non-ductile RC frame building geometries are based on the type of buildings in the building stock and prevailing architecture in Ghana. Typical structures used as hospital blocks, residential houses and government offices are generally symmetrical and regular in both plan and elevation.

The generic frames were designed according to the provisions of BS 8110 (1985) to represent archetypical RC frame structures designed in the 1980s. The choice of this code of practice was justified by the fact that it is widely used in the UK and other Commonwealth countries such as Ghana for the non-seismic design of RC frame buildings, and that very little research work has been done worldwide on the vulnerability of buildings designed to this code. The geometry of the frame RC buildings is shown in Figure 1.

The buildings were designed to a dead load of 5.0 kN/m^2, live load of 2.5 kN/m^2 and a basic wind speed of 36 m/s. In the event of the wind load being negligible, a notional horizontal load of 1.5% unfactored dead load is permitted by the BS 8110 code of practice. A mean unconfined concrete compressive strength of 20 N/mm^2 and mean yield strength of steel of 375 N/mm^2 measured for concrete cubes and steel reinforcement respectively in laboratory tests (Kankam and Adom-Asamoah, 2002).

These tests were undertaken in commercial laboratories over a period of 20 years and deemed representative of RC materials used in Accra, the capital of Ghana.

Fragility-based assessment of non-ductile RC frame buildings

A fragility curve is a graphical relationship between the probability of exceedance of a damage state and an earthquake intensity measure (IM), for example, PGA or spectral acceleration at the first period of a structure Sa (T_1). Simple fragility curves have been produced in literature for the prediction of seismic risk to RC concrete frames. The curves give an idea of the proportion of a building stock that may reach a certain limit state for a given input motion. Fragility curves are derived from vulnerability functions that are mostly defined by the characteristics of the data such as the measures of position (for example, mean, median, log median or mode) and the measures of variability (for example, standard deviation or log standard deviation). Some of the fragility curve functions employed in previous analyses (ATC, 1985; Orsini, 1999; Pasticier, 2008; Erberik, 2008; Cimellaro et al., 2009) are the normal distribution and lognormal distribution. Others (Coburn and Spence, 2002) have derived cumulative function curves that do not fit any defined probabilistic distribution using regression analysis. Rossetto and Elnashai (2003) classify vulnerability curves of RC frame buildings into four generic types according to the source of damage data. These are the empirical, judgemental, analytical and hybrid. The empirical curves are derived from the post-earthquake distribution of damage in RC frames. The limitations of this approach are that the intensity of ground motion derivation is dependent on the level of damage suffered by the buildings, and the fact that damage classification is subjective especially within the lower damage regime.

The judgement-type curves (ATC, 1985) were obtained from the opinion of earthquake engineering experts who classify building stocks into different percentages that fall into different damage classes. Rossetto and Elnashai (2003) question the reliability of this method as a result of varying expert opinion. Analytical curves (Dimova and Hirata, 2000) are derived from damage distributions of simulated damage data by either nonlinear quasi-static or inelastic time history dynamic methods or both. This involves a lot of computational effort and modelling deficiencies in an attempt to reduce variability of damage estimates.

A hybrid-type vulnerability curve attempts to improve on the disadvantages of the 3 sources of damage data by combining them in order to obtain a vulnerability curve. Examples of this approach are found in ATC-13 (1985) and ATC-40 (1996) in which empirical and expert type data were used.

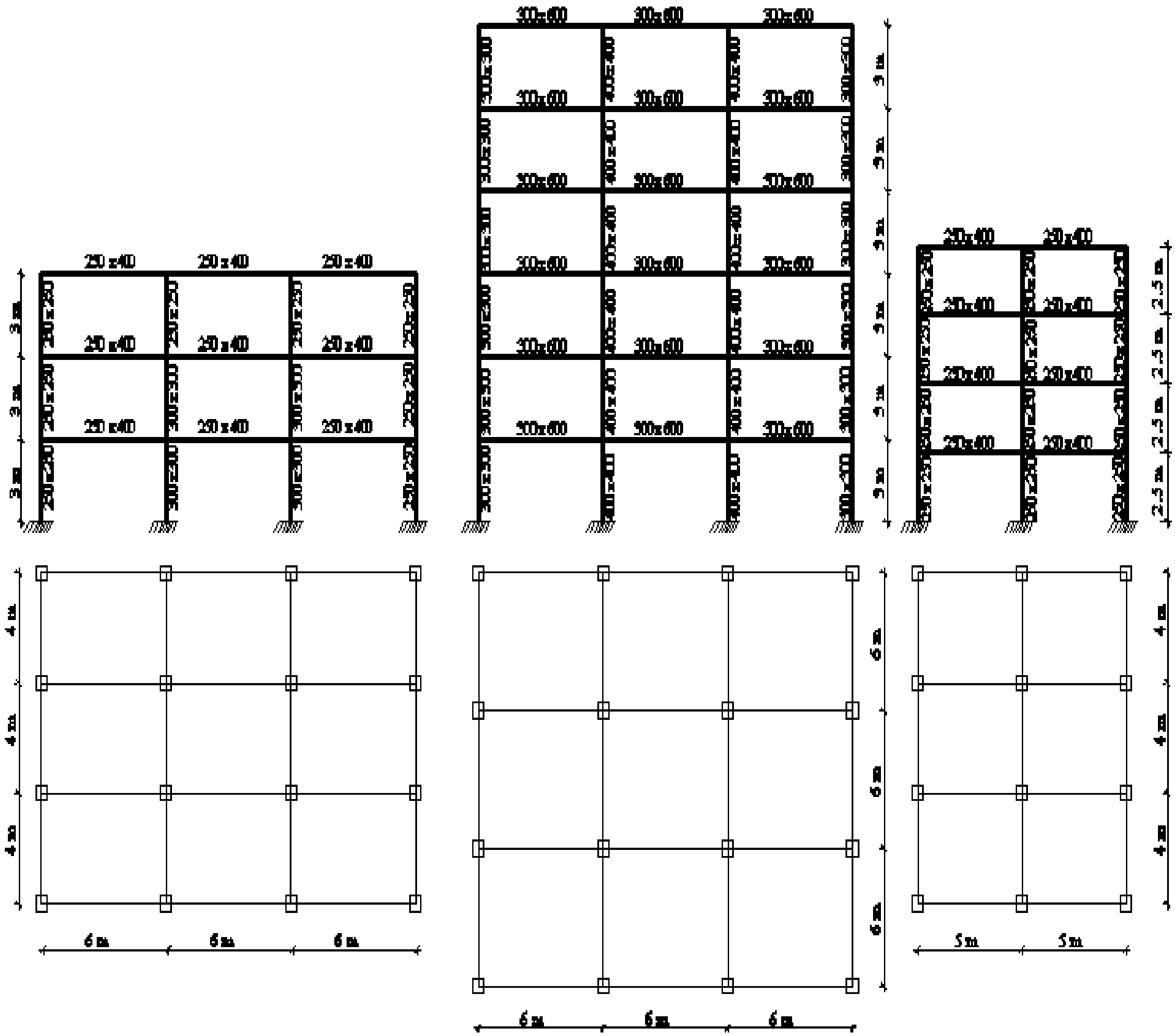

Figure 1. Geometry and dimensions of generic RC frame buildings.

METHODOLOGY

A methodology for modelling the seismic vulnerability of moment resisting non-ductile RC frames using fragility curves is summarised as shown in Figure 2. This is obtained from the study of several research works made on the seismic vulnerability of building stocks worldwide.

The major issues in modelling the seismic vulnerability of buildings are selection of earthquake intensity indicator, computational model of structure and a model for defining damage. Wen (2001) reported that in work by other researchers (Sues et al., 1985; Cornell, 1996), the uncertainty in the capacity of structures is estimated to be in the region of 40% whilst that of loading is about 80%. Others (Kwon and Elnashai, 2006) noted that the effect of randomness in material variability is far less than the effect of strong-motion characteristics.

As a result of these findings, the uncertainty due to the capacity of structural components and global structural capacity was reduced by the choice of an appropriate inelastic component model and material strengths. To ensure this, mean material properties and inelastic macro-models (Valles et al., 1996) with parameters complying with verified non-ductile buildings (Bracci et al., 1992a, b, c) were deemed adequate to represent the generic Ghanaian RC frame structures.

This is because the deficiencies observed in studies of non-ductile frame buildings done in the US were similar to those found in Ghana. Dynamic input motions employed may be real records for areas where seismic activity has been monitored or are derived from spectrum compatible time history records in areas where records are not available.

An inelastic dynamic analysis was undertaken for the structural model using the input time history records. The vulnerability of a structure or group of structures is established by deriving fragility curves.

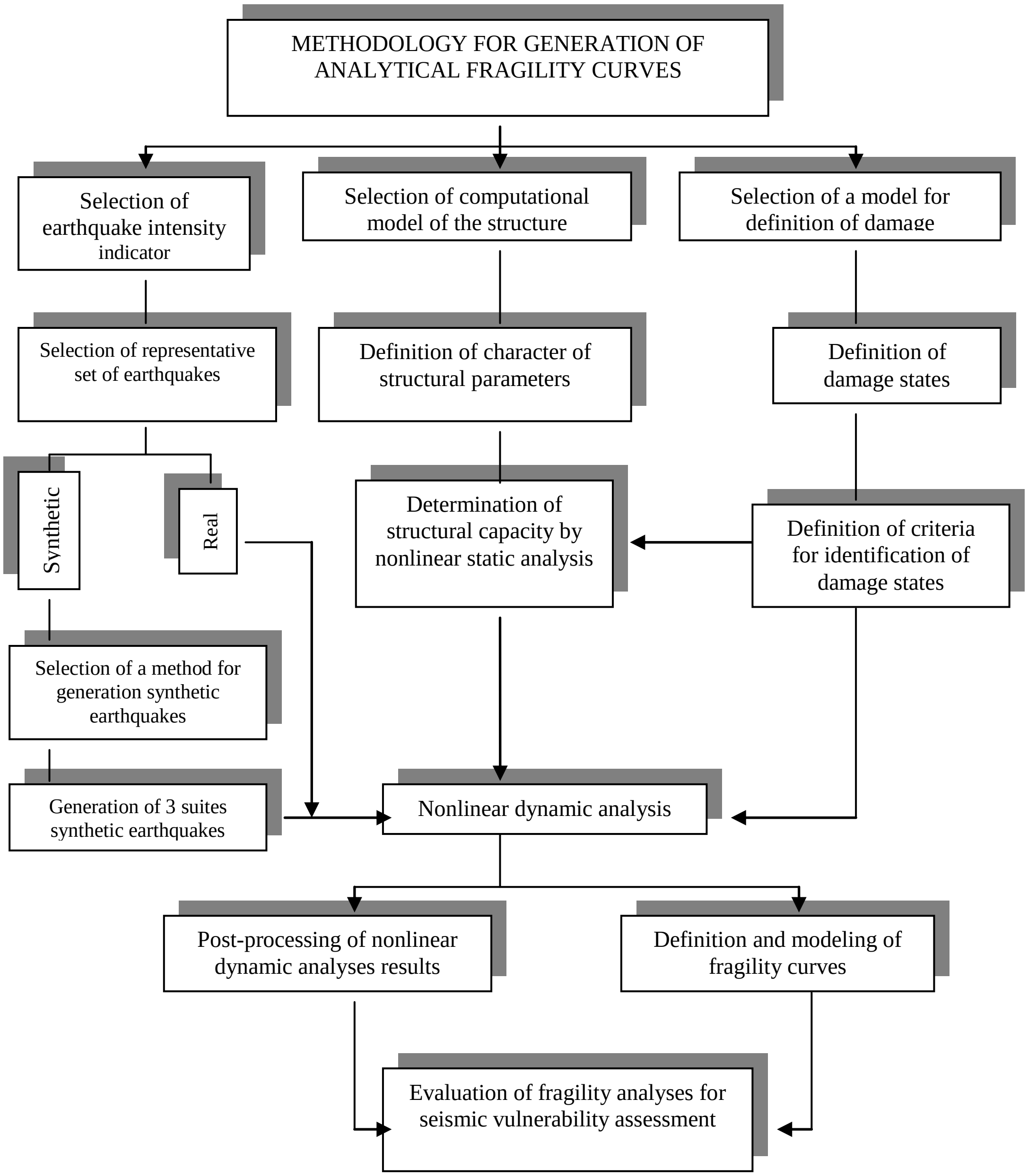

Figure 2. The steps of the proposed methodology for fragility modelling.

Structural modelling

The challenge in the analysis of non-ductile RC frame buildings is to model adequately their structural properties. This may be difficult to achieve, especially in situations where there are no laboratory component tests. In such instances, it is expedient of the researcher to use numerical analysis software that is known to have had extensive validation against laboratory testing of structural systems and components types that are of interest. One such program that was specifically developed for the inelastic static and dynamic analysis of non-ductile RC frames is the IDARC2D (1996) macro-element program. The analytical model represents material nonlinearities in beams, columns and large deformations (p-delta) effects necessary in simulating collapse. Inelastic beam and column ends are modelled with concentrated springs idealized by a trilinear backbone curve and associated hysteretic rules. The structural properties of RC members that are necessary to model a building are (i) initial stiffness, post-cracking stiffness and post- yielding stiffness, (ii) cracking and yielding bending moment capacities, (iii) equivalent viscous damping properties due to micro-cracking during elastic deformations and (iv) hysteretic properties due to inelastic deformations.

The structure was modelled using 35 and 60% of the gross sectional areas of beams and columns respectively. Essentially, the behaviour of the frames were dominated by their first mode and fundamental period of the buildings obtained from analysis were 0.73, 0.67 and 0.84 s for the 3-storey, 4-storey and 6-storey frame buildings respectively. The structural models do not include any contribution from non-structural components or from gravity-load resisting structural elements that are not part of the lateral resisting system.

Selection of a model for definition of damage

In order to achieve a practical fragility model, a parameter for expression of damage is needed. The original Park-Ang damage model (Park and Ang, 1986) modified by Kunnath et al. (1992) and introduced in the IDARC2D program was considered for the assessment of the buildings. This damage model combines both extreme deformations and the dissipated hysteretic energy at lower level of deformations. Researchers (Aycardi et al., 1992; Bracci et al., 1992c; Shahrooz and Moehle, 1990; Dumova-Jovanoska, 2000) have shown in both analytical and experimental work that in the modelling of existing non-ductile RC frame buildings, this model provides a good combination of accuracy and simplicity. The Park-Ang damage model for a structural component is defined as:

$$DI_{P\&A} = \frac{\delta_m}{\delta_u} + \frac{\beta}{\delta_u P_y} \int dE_h \qquad (1)$$

where δ_m is the maximum experienced deformation; δ_u is the ultimate deformation of the component; P_y is the yield strength of the component; $\int dE_h$ is the incremental hysteretic energy absorbed by the component during nonlinear dynamic loading; and β is a model constant parameter, which is the same as the strength degradation parameter employed in the multi-parameter hysteretic model. Their hysteretic model uses 4 basic parameters that form part of the Park-Ang damage index implemented in IDARC2D. The parameters used are: (i) a stiffness degrading parameter, (ii) a ductility-based strength deterioration factor, (iii) a hysteretic energy-based strength deterioration factor and (iv) a target slip factor. The Park-Ang damage model defined above for individual elements is extended to the global index of damage for a building. This is necessary because non-ductile RC frame buildings can undergo global damage at both story level and structural level. The global damage index is defined by averaging the damage indices of individual members such that the damage at story or structural level can be taken into account.

Determination of structural capacity

Inelastic static (pushover) analysis of the generic non-ductile RC frame buildings was undertaken. The strength and ductility capacities of the frames may actually vary depending on the gravity load combination available at the time of a seismic (in this case lateral load application) activity and the characteristics of the seismic load (different lateral load pattern). The relationship between load combinations, load patterns and structural capacity was investigated using; 3 load combinations and 4 loads patterns. The 3 load combinations used for design load estimation were; serviceability (1.0*Dead Load+1.0*Live Load), ultimate load combination (1.4*Dead Load+1.6*Live Load) and the most probable seismic load (1.0*Dead Load+0.3*Live Load). The 4 load patterns used to investigate the behaviour and structural capacity of the generic buildings were the inverted triangular (code), uniform, modal and adaptive lateral load patterns. The inelastic pushover analysis which was implemented through IDARC2D was terminated when any of the following limiting global conditions (for storey or structural level) was reached:

(1) A collapse mechanism due to plastic hinges was formed.
(2) The global yield base shear of the structure degraded below 85% of the capacity.
(3) The global Park-Ang Damage Index (DI) = 1.0 at storey or structural level.

The results of the inelastic pushover analysis performed on 3-,4- and 6-storey non-ductile RC frame buildings has been reported in another study (Adom-Asamoah and Taylor, 2006). It resulted in an additional global collapse condition of maximum inter-storey drift limits of 1%, which was implemented in the dynamic analyses post-processing model. Typical capacity envelopes of the reference buildings obtained from inelastic pushover analyses for the most probable seismic loads are shown in Figure 3a, b and c.

The 3-storey RC frame collapsed as a result of formation of a plastic mechanism when subjected to all load patterns (adaptive, code, modal and uniform). A sudden post-peak drop in base shear was observed in the capacity curves for the uniform, code and modal load patterns subjected to the generic 4- and 6-storey RC frame buildings. This was due to the increased importance of p-delta effects as a result of taller and more flexible buildings.

Spectrum compatible input time history and dynamic modelling

The selection of appropriate time-histories is a fundamental step of seismic risk evaluation (Pagliaroli and Lanzo, 2008). Artificial spectrum-compatible accelerograms were generated using the digital signal processing (DSP) program (Taylor, 1994). The advantage of the artificial spectrum compatible approach is that artificial records are obtained matching the acceleration time-series to the entire elastic spectrum. However, it is now widely accepted (Buratti et al 2010; Backer and Cornell, 2006) that the use of such artificial records is problematic, especially for non-linear analyses. The basic concern with spectrum compatible artificial records is that they generally have excessive number of cycles of strong motion. As a consequence, they possess unreasonably high energy and frequency contents. This effect has been partially reduced by adopting amplitude envelope functions in the DSP program.

The last major destructive earthquake that occurred in Ghana (22nd June 1939) was of a short duration (approximately 20 s) and an estimated magnitude of 6.5 on the Ritcher scale. The seismicity of Accra is of an intra-plate fault system that is typically characterised by short durations (for example 10 to 20 s). Therefore, 3 response spectral accelerations were derived on rocky ground conditions for this study. The UK, EC8 and HNK response spectra were used to generate an envelope of spectra records to be used for inelastic dynamic analyses. As Ghana's seismicity arises from intra-plate mechanisms similar to the UK, it was decided to use UK parameters in lieu of Ghana specific parameters. The EC8 response spectrum was also adopted to represent typical European strong motion records. The HNK spectrum is representative of a composite (multi-site) three peak response spectrum which is deemed necessary by the author under current circumstances and restrictions.

The DSP program was used to generate short period time history records of 10.24 s duration for the UK, and EC8 spectra compatible records whilst the composite HNK spectrum compatible records were of duration 16.8 s. Given the nonlinear nature of the problem, it was recognised that the analyses had to be based on a reasonably large set of input acceleration time histories. The influence of the number of time histories was studied and the number of time histories necessary to assure statistical reliability was about 30. However, 100 spectrum compatible time histories were generated for each spectrum type. The mean curves of the three suites of 100 spectral pseudo-acceleration spectra are shown in Figure 4.

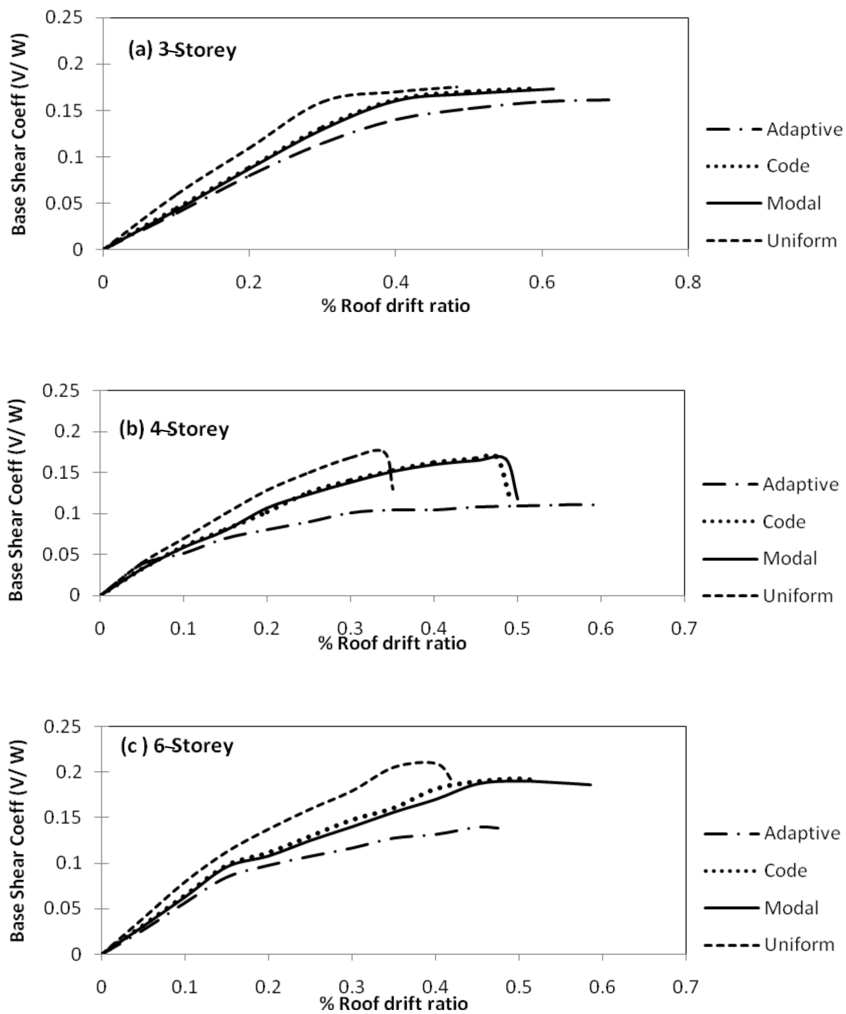

Figure 3. Capacity curves of the 3 generic RC frame buildings obtained from inelastic pushover analysis.

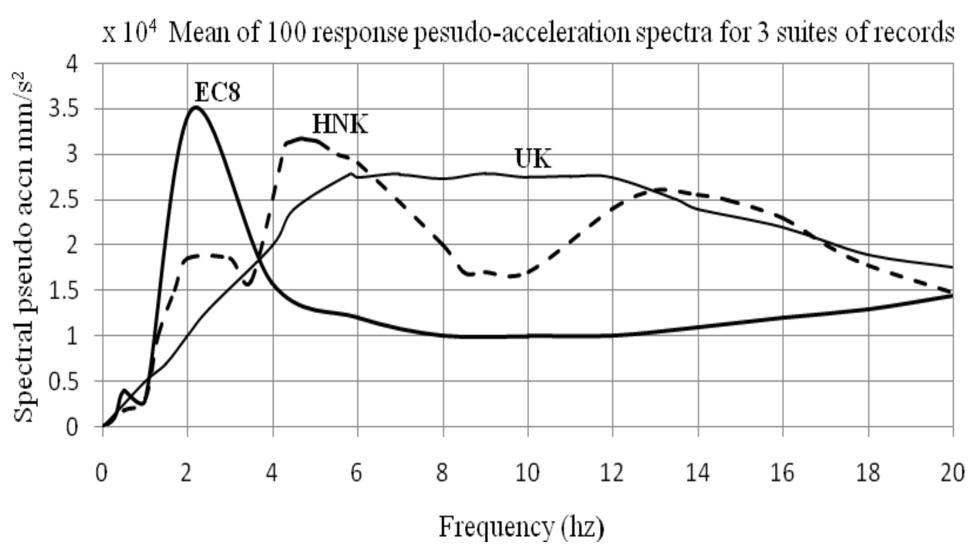

Figure 4. Mean of 100 Spectral acceleration response diagrams of EC8, HNK and UK records.

Generation and analyses of fragility curves

A limited Monte Carlo type fragility (vulnerability) analyses was performed for this study using a probabilistic approach. As already established, the most significant level of variation in the response of the buildings was therefore assumed to be due to input time histories. The three (3) suites of spectrum compatible time history records (EC8, HNK and UK) each of 100 accelerograms were applied to the buildings. Inelastic dynamic analyses (IDA) were performed by gradually scaling the ground motion intensity of a record until building collapse was achieved. For each ground motion, collapse was defined on the basis of 2 intensity measures; Sa (T_1) (spectral acceleration at the first-mode period of the analysis model) and PGA. PGA is found not to be a good intensity measure because the same magnitude of PGA does not mean the same frequency content, event duration and effective number of loading cycles. Therefore, the same level of PGA does not assure the same level of response and damage of a structure. On the other hand, spectral acceleration at first mode period of a structure Sa (T_1) is a measure of ground motion that takes into account the sustained shaking energy at a specific period. Studies (Vamvatsikos and Cornell, 2002; Mwafy and Elnashai, 2001; Mwafy and Elnashai, 2001; Annan et al., 2009; Shome et al., 1998, Azarbakht and Dolsek, 2007) have revealed that by scaling ground motion records to the target Sa (T_1), seismic demands at a specific ground motion seismic hazard can be efficiently estimated.

However, since the seismic hazard of Ghana is only specified in terms of PGA, both PGA and the preferred intensity measure of Sa (T_1) were used as intensity measures in this study.

The outcome of an IDA is a fragility function, a cumulative probability distribution that defines the probability of structural (simulated) collapse as a function of the ground motion intensity given in this research as Sa (T_1) and PGA. The results of the generic non-ductile RC frame types analysed here are therefore presented in the form of cumulative frequency distributions (CFD). Fragility curves in this research were based on the two-parameter lognormal distribution function. This approach was used by other researchers (Chenouda and Ayoub, 2009; Shinozuka et al., 2000; Mehanny and Howary, 2010) and found to be precise. The results of structural collapse fragility functions are discussed next.

RESULTS AND DISCUSSION

Assessment of fragility curves using Sa (T_1) as intensity measure

CFD curves fitted to the lognormal distribution with collapse Sa (T_1) conditions as the intensity measure are shown in Figure 5a, b and c. In order to account for the

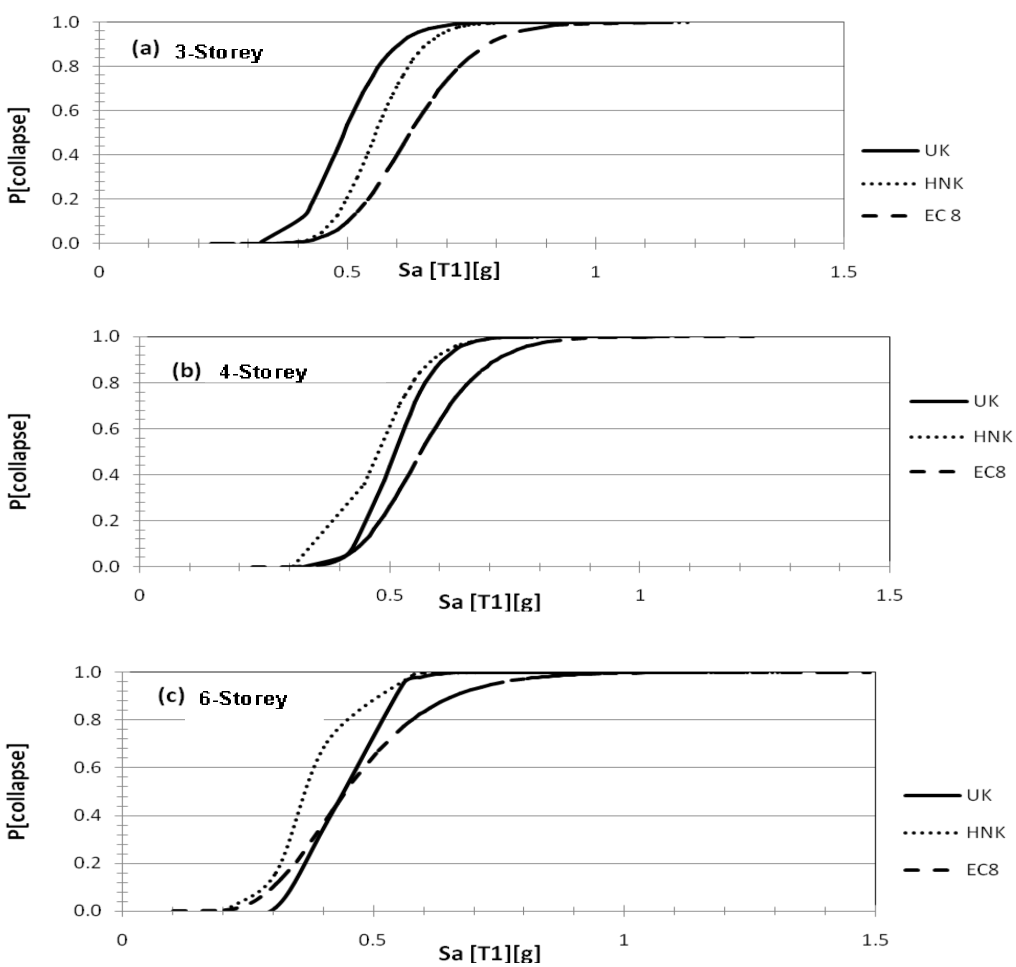

Figure 5. Fragility curves using Sa (T_1) as IM.

variability of seismic hazard, spectral acceleration Sa (T_1) values at specific structural periods have been introduced in seismic codes of practice such as the National Building Code of Canada (2005). It is as a result of this that mean fragility curves are produced using Sa (T_1) even though the proposed seismic hazard map of Ghana is based on the PGA as its' intensity measure. Since engineering designs may be based on 5% probability of collapse, 5% damped spectral acceleration values Sa (T_1) are established for 5% probability of collapse of the building types. The 5% probability of collapse values for the 3-storey building as shown in Figure 5a were 0.36, 0.42 and 0.44 g for the UK, HNK and EC8 spectra records respectively. From Figure 5b, the 5% probability of collapse for 4-storey buildings subjected to the UK, HNK and EC8 spectra records were 0.40, 0.32 and 0.40 g respectively. The 6-storey building is expected to have a 5% probability of collapse Sa (T_1) of 0.32, 0.24 and 0.26 g for the UK, HNK and EC8 records respectively as shown in Figure 5c. The effect of the energy content of the spectral acceleration in the region of the fundamental period of the buildings was not evident. It was expected that the EC8 records produced the lowest spectral building types. This is as a result of the concentration of high energy in the EC8 spectrum around the regions of the fundamental periods of the buildings (Figure 4).

Lower acceleration spectral values refer to collapse occurring at lower intensity levels. No trend was observed in the failure intensities of a particular generic frame building subjected to the 3 suites of artificial time histories. However, the collapse intensities of the 6-storey frame buildings were lower as compared to those of the 3- and 4-storeys for all the records. This could be due to the lessened influence of the first mode, and the stronger effect of higher mode contribution to response in the 6-storey buildings which coincides with the peaks of the spectral accelerations, so ability of building to collapse under lower earthquake intensities.

Assessment of fragility analysis using PGA as intensity measure

CFD curves fitted to the lognormal distribution with collapse PGA condition as the intensity measure are shown in Figure 6a, b and c. The discussions here will be

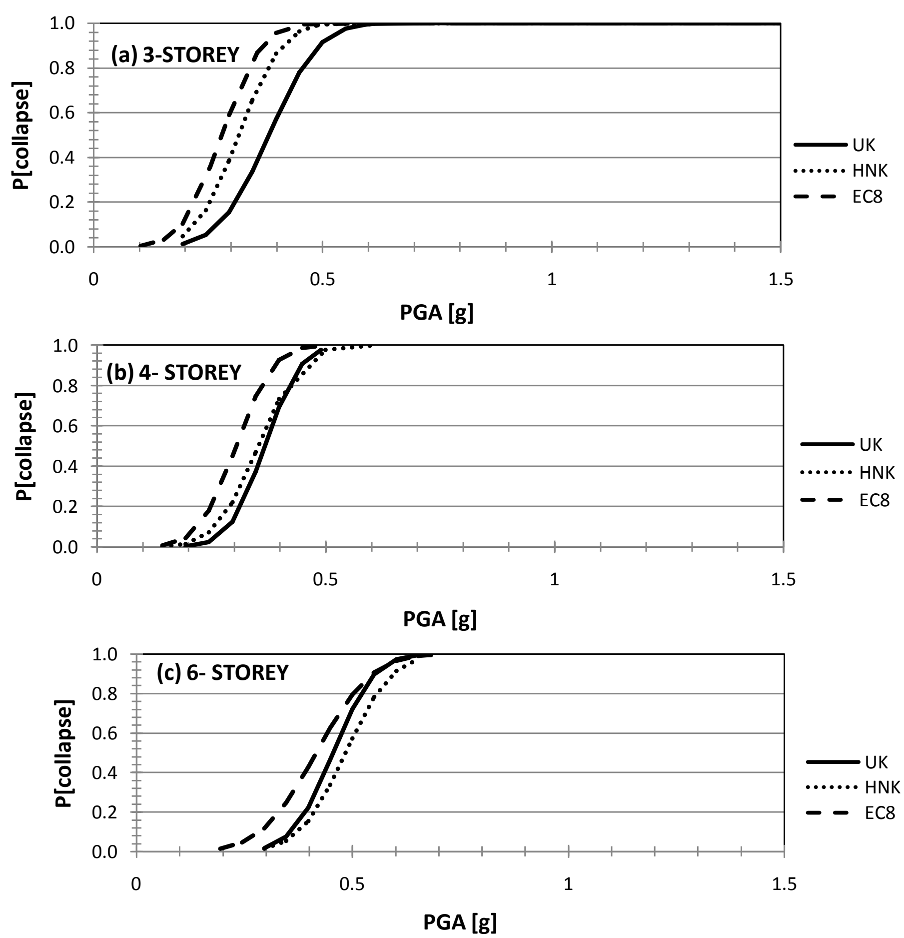

Figure 6. Fragility curves using PGA as IM.

based on the probability of the building types surviving specific PGAs. Among the generic 3-storey buildings, the probability of suffering collapse as observed in Figure 6a, were 35, 70 and 85% for PGA level of 0.35 g for the UK, HNK and EC8 spectra respectively. Liao et al (2006) reported fragility functions of general building structures in Taiwan by sampling damage rates of different damage states during the 1999 Chi-Chi Earthquake. For low-rise (1-3 stories) concrete frame structures subjected to a measured PGA of 0.325g in Tsao-Tung, investigations showed that 29% of the buildings suffered complete collapse. This damage rate was higher than that derived from analytical fragility studies by the authors. Higher damage rates of 35-85% obtained in this study of Ghanaian frames subjected to PGA of 0.35g may be justifiable. This is because whilst the buildings in this study were designed to a notional horizontal load of 1.5% dead load, the buildings used by Liao et al. (2006) were designed to moderate seismic code capacity of 0.1g. The probabilities of the generic 3-storey buildings suffering collapse for PGA level of 0.25 g were 6, 20 and 35% for the UK, HNK and EC8 spectral records respectively. The probabilities of collapse during a PGA level of 0.35 g event for 4-storey buildings subjected to the 3 spectral records (Figure 6b) were 40, 50 and 75% for the UK, HNK and EC8 spectra records respectively. The percentage of the generic 4-storey frames buildings that may suffer collapse were 5, 13 and 21% for the UK, HNK and EC8 spectra compatible records respectively. Among the generic 6-storey buildings whose fragility curves are shown in Figure 6c, the probabilities of collapse in a PGA level of 0.35 g were 10, 7 and 26% for the UK, HNK and EC8 spectra records respectively. The probability of suffering collapse for the generic 6-storey building subjected to EC8 spectrum records (Figure 6c) was 6% for PGA level of 0.25 g.

The high probability of collapse of the generic non-ductile RC buildings expected to collapse under EC8 for each category of building could be explained as a result of its high energy concentration (Figure 4) around the typical periods of 3-6 storey buildings. The UK spectra records which have lower energy levels in the frequency ranges of the selected buildings derived lower probability of collapse compared to those of the HNK and EC8 spectrum compatible time histories.

Conclusions

An analytical study of the seismic vulnerability of non-ductile RC frame buildings designed to BS 8110 was undertaken. A methodology for assessing the fragility curves were derived for the 3-, 4- and 6-storey generic buildings conditional on the 3 generic near-fault like response spectra with duration of about 10 to 16 s. Cumulative frequency distribution curves were fitted to the lognormal distribution and the fragility function parameters established for the collapse conditions. The seismic intensity measures of interest were PGA of the input motion and spectral acceleration at the fundamental mode period of the buildings Sa (T_1) that caused the collapse of the buildings. In order to obtain meaningful rule of thumb values for the short duration near-fault like records, mean probability of collapse intensity values were established for the generic buildings. Typical generic non-ductile 3 to 6 storey RC frame buildings subjected to near-fault like ground motions may record high probabilities of collapse if they are situated in 0.25 to 0.35 g PGA seismic zones. Fragility curve information was also used to establish 5% probability of collapse values of Sa (T_1) of the generic buildings. It is important to note that this study and the conclusions thereof are so far valid for symmetrical 3 to 6 storey non-ductile moment resisting bare RC frame buildings in Ghana. Extrapolation of the results presented herein to either irregular frames or in filled frame buildings shall be the subject of a similar effort before being either applied or denied.

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