Assessment of Collapse Capacity of RC Buildings Based on Fiber-element Modelling

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ABSTRACT: Despite the fact that structural collapse is a key contributor to seismic risk, robust procedures for the assessment of structural collapse are typically absent in seismic design documents. This paper discusses probabilistic collapse capacity assessment of reinforced concrete buildings using the New Zealand red book building as a case study. Load resisting elements of RC buildings are traditionally modelled using lumped plasticity elements at predefined plastic hinge locations which require crude approximations of several features of post-peak response, which is very important in simulating structural collapse. This paper presents a discussion on the use of fiber-based nonlinear modelling, which utilizes generic path-dependent cyclic stress-strain relationships of concrete and reinforcing steel, for prediction of RC frame building collapse. A collapse capacity distribution accounting for ground motion uncertainty is estimated using a suite of ground motions and conducting incremental dynamic analysis (IDA), i.e. gradually increasing the intensity of the ground motions until the occurrence of collapse. It is shown that the fiber model simulates the collapse mechanism of the building without the upfront assumptions required in lumped plasticity modelling; thereby making the prediction more reliable.

1 INTRODUCTION

While collapse safety of buildings is a key contributor to seismic risk, methods to quantify collapse potential of buildings are absent in design documents. Following advancements in computational power and observational and experimental validation of analytical models that capture a structure’s behaviour as it approaches collapse; numerous attempts to quantify collapse potential of buildings have been performed within the last decade. Quantification of collapse risk is commonly assessed using Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell 2002) in which structures are subjected to ground motions with increasing intensity until the occurrence of collapse. Following the introduction of PEER probabilistic performance assessment framework (Moehle and Deierlein 2004), collapse of structures was introduced as part of conceptual performance-based design (Krawinkler et al. 2006, Zareian and Krawinkler 2007). Since then, in a quest of more accurate estimation of structural collapse, several aspects of building collapse have been studied. In general, three main aspects related to improving collapse risk of buildings have been scrutinised in literature; structural modelling to simulate collapse, proper ground motion selection, and quantification of uncertainties in the assessment process.

As for the structural modelling aspect of collapse assessment, which will be the focus of this paper, Ibarra and Krawinkler (2005), for example, proposed a lumped plasticity component hysteresis model that accounts for strength deterioration in backbone curve as well as strength and stiffness deterioration in cyclic loading. Following the initial introduction of this particular lumped plasticity model, several further improvements and calibrations have been proposed (e.g. Haselton and Deierlein 2007, Lingos and Krawinkler 2012).

Despite significant progress in development of structural models to simulate structural collapse under earthquake loading, most of the research conducted so far has been based on lumped plasticity models, which bring about several approximations and limitations. For example, collapse/failure modes due to brittle shear failure of RC sections and axial load effects on moment capacity of structural elements throughout the loading are typically neglected in these models. In the lumped plasticity models, developed in particular following Ibarra and Krawinkler (2005), axial loads can only be considered in terms of their $P - \Delta$ effects, which may result in dynamic instability of buildings. In fact this is the only form of collapse which is considered in these models. Collapse due to the inability of structures to carry gravity loads is not taken into account. Furthermore, plastic hinge models require a rough assumption of plastic hinge location along the element length. In addition, lumped plasticity models have to be calibrated for each individual section which adds further errors in the assessment procedure.

This study aims to illustrate the potential for improved structural modelling of collapse simulation by implementing fiber-based modelling of RC elements. The fiber model addresses most of the limitations of typical lumped plasticity models by: (i) explicitly considering loss due to vertical load carrying capacity; (ii) eliminating the need to consider plastic hinges, i.e., plastic hinge location and length; and (iii) generation of the structural model based on material properties rather than calibration at the section level. A case study building is used to demonstrate the use of the fiber-based model for collapse potential assessment. Pushover analysis is carried out to understand the collapse mechanism and then collapse capacity of the building is computed through a conventional IDA.

2 COLLAPSE MECHANISM OF RC ELEMENTS

Collapse of a building occurs due to local and/or global failure of sufficient number of structural components to cause instability. At the component/element level, failure occurs due to failure of a critical section, which is intern the result of strain material failure.

In an RC element subjected to monotonically increasing flexural action, cover concrete is first crushed and removed off the critical section once strain in the unconfined concrete equals crushing strain. Following the loss of cover concrete, the section resistance will rely increasingly on the confined core concrete in addition to the longitudinal reinforcing bars. If the compression bars are not well restrained against buckling, or if adequate transverse reinforcement is not provided, the critical section may fail in shear or buckling of longitudinal bars may occur before the core concrete reaches its crushing strain. In contrast, in a well confined section, confinement of core concrete allows longitudinal tension bars to enter the strain hardening region. In such sections, failure of the section eventually occurs due to failure of the core concrete. If the section is over reinforced, this failure occurs before yielding of longitudinal bars, whereas in under-reinforced sections, yielding of longitudinal bars takes place before crushing of the core concrete.

In cyclic loading of a well confined section, in addition to the failure modes induced by monotonic loading, failure may also occur due to strength deterioration of the cracked concrete and longitudinal bars which can be caused by buckling of compression bars as well as fatigue of steel bars before core concrete reaches its ultimate strength.

This study aims to provide a structural model that enables simulation of collapse mechanism more realistically (i.e. consistent with the deterioration of materials observed in experiments). In a fibre-based nonlinear structural model, collapse of a section occurs because of the failure of the majority of fibres in the section. Based on the discussion above, a proper fiber model for simulating collapse has to contain material properties that can capture crushing of cover concrete and core concrete based on the confinement configuration, and strength degradation of reinforcing bars in cyclic loading caused by buckling and fatigue. In the following sections, constitutive material models to capture these characteristics will be discussed. The material models are then used to assess collapse potential of a case study RC moment frame building.

3 FIBER MODELLING FOR COLLAPSE ASSESSMENT

3.1 Nonlinear fiber-element modelling

Fiber-element nonlinear modelling is a microscopic modelling technique in which each element is represented using a single line. Implementation of a fiber model is schematically illustrated in Figure

1. The member cross section is divided into many cells (Figure 1b). The strain of each cell is calculated based on the Euler-Kirchoff's hypothesis, i.e., plane section remains plane after bending. For each fiber strain along the axis of the element, response is calculated using the material constitutive models representing the local behaviour (figure 1c). The accuracy of a fiber-section model is almost entirely dependent on the ability of constitutive material models to represent the overall inelastic behaviour of the member. Therefore, rather than calibration of structural components at section level in lumped plasticity models, material characteristics have to be carefully chosen to represent true structural behaviour. Degrading behaviour of the elements is also influenced by the initiation of buckling in the longitudinal steel, which may lead to fracture of transverse reinforcement and loss of confinement.

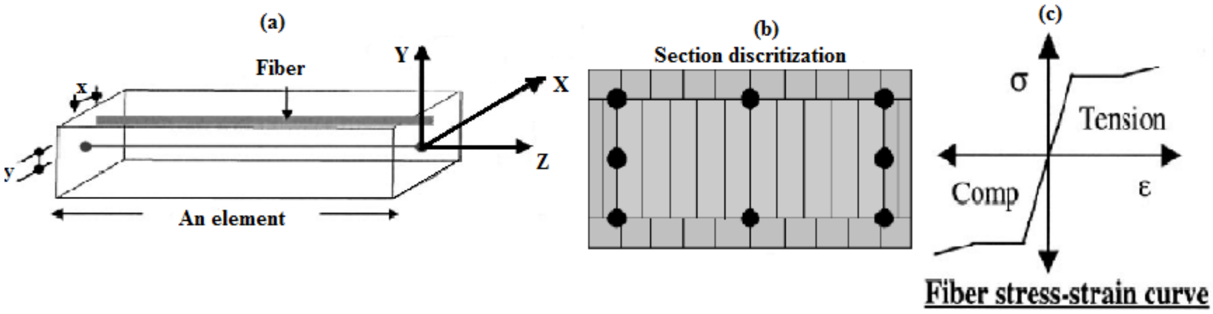

Figure 1 Schematic illustration of fiber-element modelling of RC elements (a) An RC element (b) section discretization (c) stress-strain curve of an example fiber

3.2 Adopted fiber models of concrete and steel

3.2.1 *Confined and unconfined concrete*

In order to simulate collapse of RC buildings, PEER's open source structural analysis and simulation tool OpenSees (2012) is utilized herein. In OpenSees, several material properties have been implemented which enable simulation of structural collapse. In this study, Popovics (1973) concrete model is used for cyclic behaviour modelling of confined and unconfined concrete. Figure 2a illustrates the stress-strain envelope of the concrete fibers used in the study. This model requires specification of seven parameters to control monotonic behaviour of the model for both confined and unconfined concrete: concrete compressive strength in 28 days f_c, concrete strain at maximum strength ε_c, concrete strain at crushing strength ε_{cu}, initial stiffness E_c, maximum tensile strength of concrete f_{ct}, ultimate tensile strain of concrete ε_t, and an exponential curve parameter to define the residual stress β (optional). Parameters of the model can be obtained by a simple uniaxial experimental test. Alternatively, for practical purposes, compressive strength for unconfined concrete in 28 days can be obtained from the design documents of the structure. Mander et al. (1988) have proposed a widely accepted theoretical stress-strain model for concrete, depending on the confinement configuration, which can be used to quantify the maximum stress and strain of confined concrete. Approximate equations have been suggested in building codes for initial stiffness of concrete. If $E_c = 4734\sqrt{f_c}\ (in\ MPa)$ is used to estimate initial stiffness, then Popovics envelope model will be identical to the model proposed by Mander et al. (1988).

For collapse prediction, concrete strain at crushing is the most important parameter in defining the concrete stress-strain model. As will be shown later in the paper, collapse capacity of the case study RC building is directly dependent on this value, which is controlled by the confinement configuration of critical sections. Theoretical values for strain at the crushing point have been proposed in literature depending on the number, size, and distance of transverse bars in the section which can be used in the model (e.g. Karthick and Mander 2011). Based on the proposed values in literature and experiences of the authors, a strain value of ε_{cu}, =0.03 will be used in this study for crushing of confined concrete detailed as per New Zealand concrete standard NZS3101.

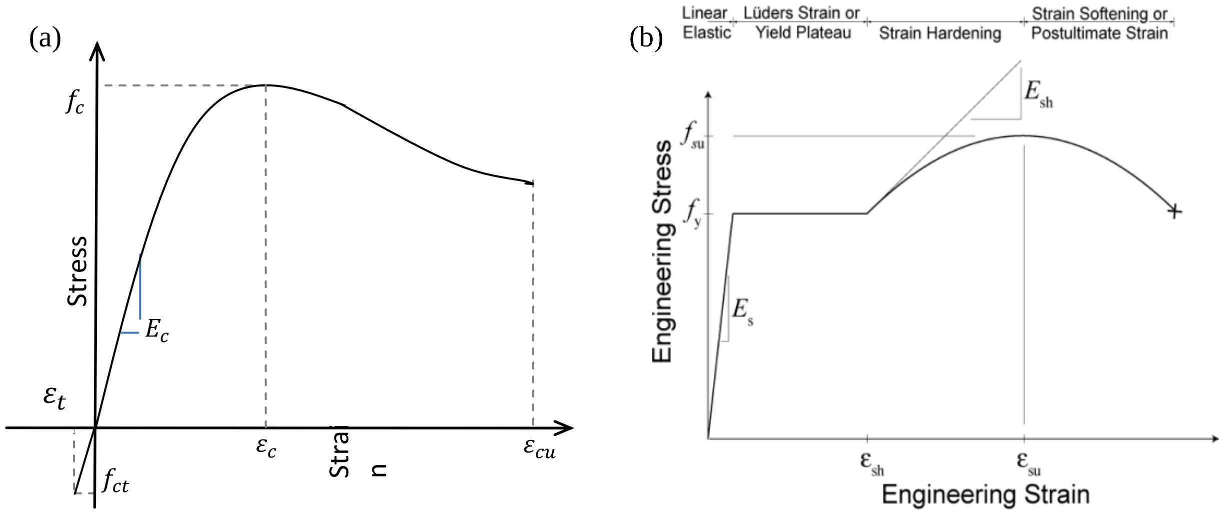

Figure 2 (a) Cyclic tension-compression envelope of Popovics model (b) Stress-strain envelope curve of the steel model used in the study (figure from the OpenSees (2012) website)

3.2.2 *Reinforcement steel model*

In order to simulate steel bars in RC sections, a generic phenomenological material model developed by Kunnath et al. (2009), capturing strength degradation due to fatigue in cyclic loading as well as buckling of reinforcing bars in compression, is utilized. Figure 2b illustrates a typical envelope curve of the reinforcing steel model. The envelope curve is based on the Chang and Mander (1994) uniaxial steel model.

The degrading behaviour of RC section is influenced by buckling of longitudinal steel bars in compression. Buckling of compression bars in the model is incorporated based on the model proposed by Dhakal and Maekawa (2002a, 2002b). Fatigue parameters are based on the Coffin-Manson equation for plastic strain amplitude. The softening region (strain beyond the ultimate stress point) shown in Figure 2b is a localization effect due to necking and is a function of the gage length used during measurement. This geometric effect is ignored in this simulation. In this simulation, it is assumed that there is no softening in natural stress space. Because the simulation always converts back to engineering stress space, some softening will still be observed in the tension response due to the reduction in cross section area resulting from necking; however this will be much smaller than that shown in the original backbone curve proposed by Chang and Mander (1994).

Due to the nature of the steel model used herein, steel reinforcement failure may occur due to buckling of compression bars or fatigue of longitudinal bars after several cycles of loading, but pure tensile fracture (i.e. ε_{su}~0.1-0.2) is not modelled because the steel strain is maintained beyond the peak strain. As will be shown in later sections, this does not influence the collapse simulations because concrete fibers crush, and reach zero stress, prior to the steel bars reaching their peak strain; thereby resulting in failure of the section. A peak strain of ε_{su} =0.1 will be used in this study as a conservative estimate based on the experiences of the authors with the steel bars used in New Zealand construction industry.

4 OBSERVED COLLAPSE MECHANISM OF CASE STUDY BUILDING

In this section, the fiber element model described in the previous section is utilized to perform a pushover analysis of a case study RC frame building until structural collapse occurs. The case study building is the 10 storey New Zealand Red Book building (Bull and Brunsdon 1998), which acts as a design example of the New Zealand concrete code. Figure 3 shows the plan and elevation views of the building layout. The primary lateral load carrying system consists of four one-way perimeter moment resisting frames which are three bays long. Vertical loads are transferred primarily through interior columns with gravity beams supporting one-way floor units. Further details of the structural properties and design details can be found in Bull and Brunsdon (1998).

A fiber-based 2D model of the perimeter frame is developed in OpenSees using the material models explained in previous sections. The building has been designed such that columns in different levels have the same cross-section. Similarly, beams in various levels also share the same cross-section details. The building's first mode period amounts to T_1=1.02 sec. This period is indeed based on the pure elastic gross stiffness of the elements rather than typical cracked stiffness used in lumped plasticity models.

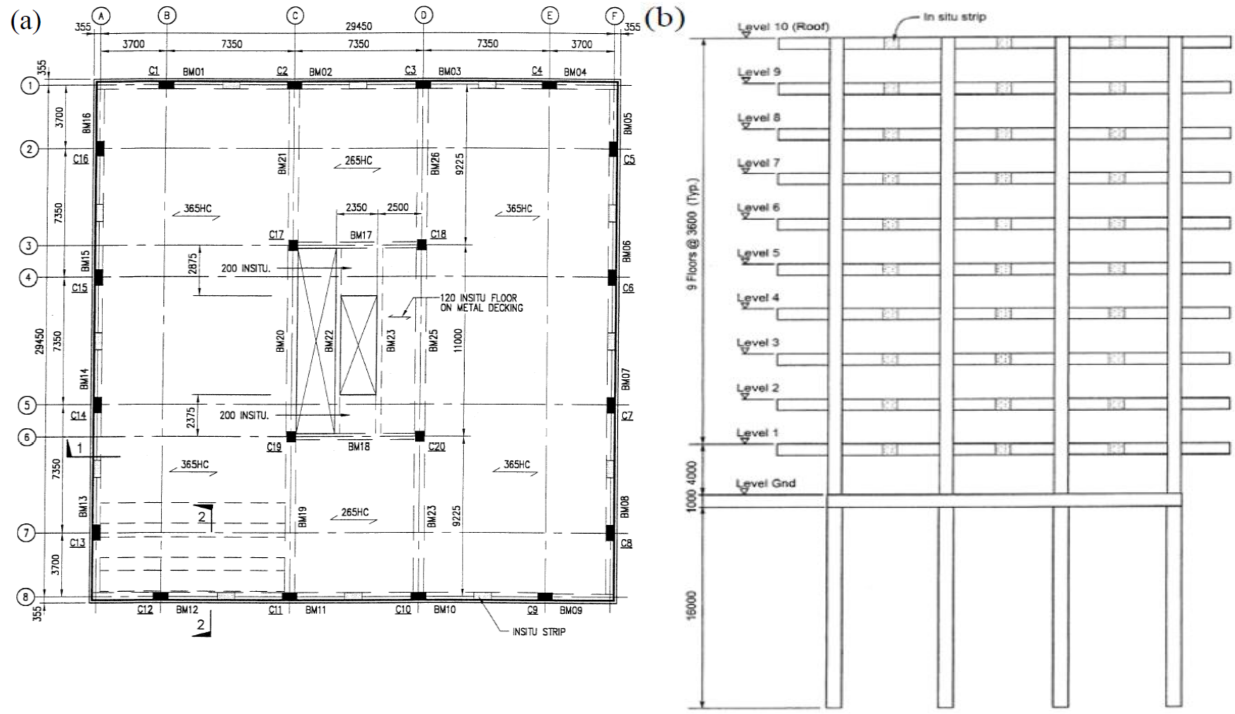

Figure 3 Plan and elevation view of New Zealand Red Book building used as the case study

Pushover analysis is performed to verify the collapse mechanism in the fiber model to predict collapse capacity. The first vibration mode shape is used as the load pattern for the pushover analysis. Figure 4 depicts the pushover curve until occurrence of structural collapse. As can be seen in the figure, the pushover curve clearly shows a gradual change in stiffness due to cracking of concrete in early stages of the loading until reaching a plateau which indicates yielding of reinforcement bars. Strength degradation in the pushover curve is then observed while critical fibers lose their strength. Finally, global collapse occurs when a sufficient number of concrete fibers in the critical sections reduce to zero stress due to concrete crushing.

The pushover-based collapse mechanism can be further scrutinized by observing Figure 5, where stress-strain histories of a confined concrete fiber and a steel bar in the critical section are shown. Figure 5ashows stress-strain behaviour of a confined concrete fiber in compression in the critical section. As can be seen, the concrete fiber fails once the concrete strain exceeds 0.03 which was specified as the confined concrete crushing strain. After this level of strain, the concrete stress drops to zero and the section starts to lose its strength. Figure 5b depicts the stress-strain history of a steel bar in tension at the same critical section. It is observed that an abrupt drop in strength of the steel bar occurs when concrete fibers lose their strength, leading the whole section to undergo compression, which is marked in the figure. Eventually, buckling of steel bars, due to the failure of compression concrete fibers at the compression part of the section, leads tofailure of the section. The failure of one section will result in the redistribution of loads to nearby elements which will lead to consequent overloading and hence cascading failure initiates. After the failure of critical sections, i.e. columns in the first floor, the building will not be able to carry gravity loads and subsequently a global collapse of the building will take place.

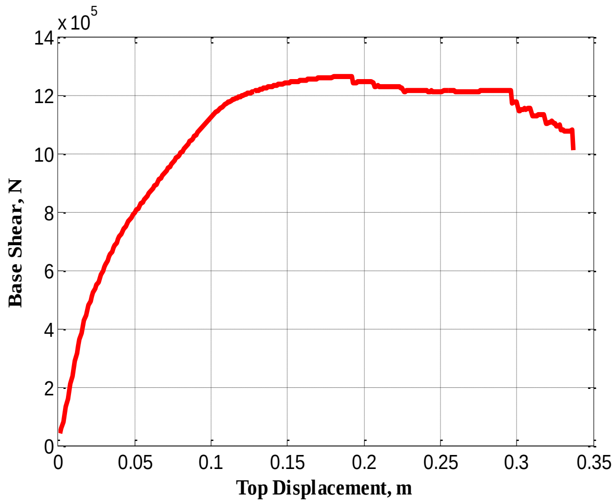

Figure 4. Pushover curve of the case study building with 0.03 for crunching strain of confined concrete

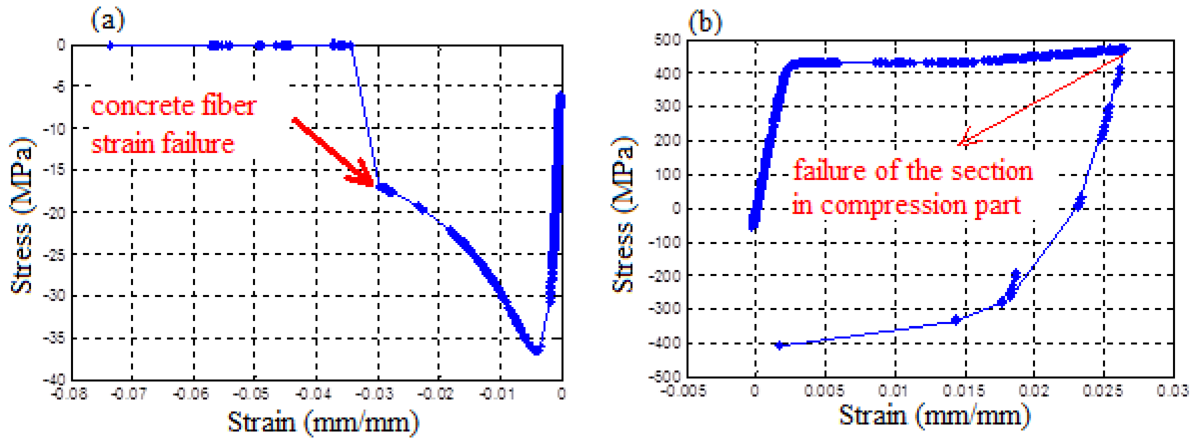

Figure 5. Stress-strain diagram of a confined concrete fibre in compression (left) and a steel fibre in tension (right) for critical section of the case study building

5 ASSESSMENT OF COLLAPSE PROBABILITY VIA INCREMENTAL DYNAMIC ANALYSIS

Probabilistic assessment of building collapse is conventionally performed by an IDA (Vamvatsikos and Cornell 2002), which is gradually increasing the intensity of ground motions until the occurrence of collapse is observed. Selection of a proper suite of ground motions forms an important part of the IDA. Several ground motion selection methods as well as alternatives for intensity measures to be used in ground motion selection have been studied. However, as the focus of this paper is to examine the use of the described fiber model in structural collapse assessment, a conventional IDA is used herein to predict building collapse capacity. A set of 20 ground motions is selected in accordance with site characteristics of the building which best match the New Zealand seismic code (NZS1170.5:2004). Among the two horizontal components of each ground motion the one with lager spectral acceleration at the fundamental period of the building is selected. Spectral acceleration at the first mode of vibration, $S_a(T_1)$ is selected as the intensity measure. Ground motions are scaled up in intervals of 0.1g until failure of the building.

In order to observe the collapse mechanism of the building in the proposed model, Figure 6 illustrates the stress-strain history of two fibers in a critical section of the right external column on the first floor subjected to one of the selected ground motions. Figure 6a shows the response history of one of the steel bar fibers for an example ground motion scaled to an intensity level which just causes collapse.

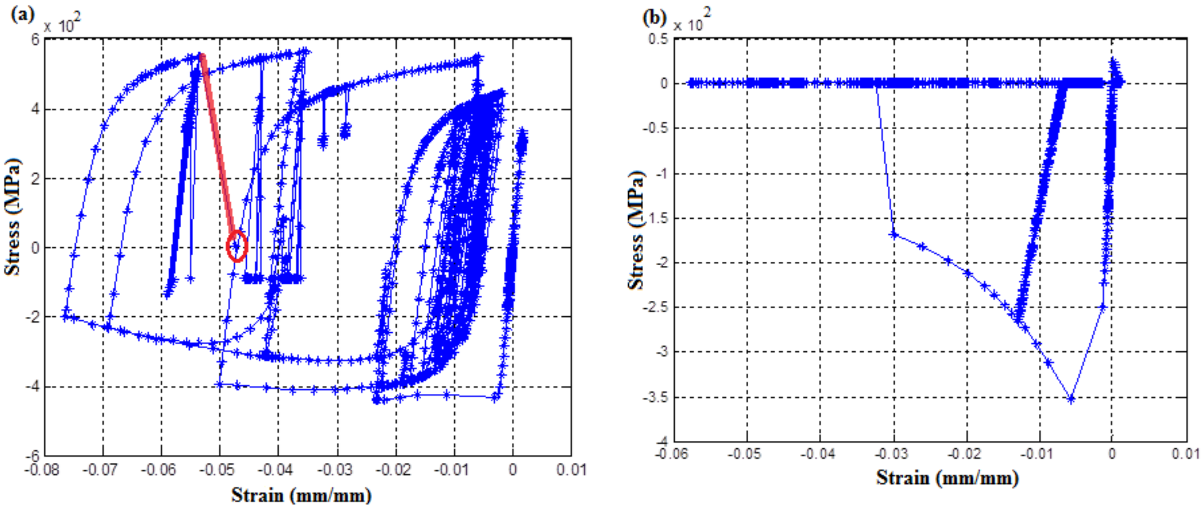

Figure 6. Stress-strain history diagram of two fibers in the critical section of the case study building subjected to an earthquake. a) steel bar fibere b) confined concrete fiber

Cyclic strength degradation of the steel bar, an important issue in collapse assessment simulation, is clearly seen in the figure. The last branch of the loading is also highlighted in the figure indicating sudden strength deterioration in the steel bar to zero due to failure of the majority of concrete compression fibers, resulting in loss of strength ofthe section; thereby rendering it unable to carry vertical loads.

Figure 6b illustrates the response history of a confined concrete fiber in compression at the critical section subjected to the same example ground motion. In this figure, small tensile stress as well as failure of the concrete fiber in compression can be observed. Despite the concrete stress in the fiber reducing to zero, the section continues to carry load because other fibres in the section are still able to carry loads. Failure of the section finally occurs when enough fibers in the section lose their strength. In fact, collapse of the building occurs because of the existence of axial loads and failure of critical sections to sustain gravity loads. This indicates that neglecting axial load effects and the potential for gravity load collapse on the section strength in traditional hinge-based models may lead to major errors in collapse estimation.

IDA curves of the building for the suite of 20 selected ground motions are shown in figure 7a. The end point of each curve shows the highest intensity (on the vertical axis) scaling of the individual record for which the building did not collapse. The cumulative distribution function, assuming a lognormal distribution, of these spectral acceleration values that correspond to structural collapse is defined as the 'collapse fragility curve' and is shown in figure 7b. It can be observed that the median collapse capacity of the building amounts to roughly 0.95g. It can be seen that at the 10% in 50 year ground motion level of Sa(1.02)=0.26g as well as for the 2% in 50 year ground motion level of Sa(1.02)=0.47 g, based on NZS1170.5:2004 for site class C, the probability of collapse is zero. However, it is noted that if the revised value of the $z = 0.3g$ for hazard estimation in the New Zealand code is used, at the 2% in 50 year ground motion level of Sa(1.02)=0.63g the probability of collapse would be around 5%.

6 CONCLUDING REMARKS

A fiber-based numerical model was developed in this paper to more accurately simulate structural collapse of RC buildings, as compared to conventional lumped plasticity modelling. The fiber model does not require upfront assumptions and calibration of structural components as in lumped plasticity models, instead requiring simple material properties, which can be obtained by simple uniaxial tests, or be advised by material providers for practical purposes. It was shown that, for the case study building considered, collapse occurs due to crushing of compression concrete followed by sudden strength deterioration of steel bars caused by buckling or low cycle fatigue. Using the fiber-based structural model, collapse capacity of a case study RC frame was then predicted via IDA.

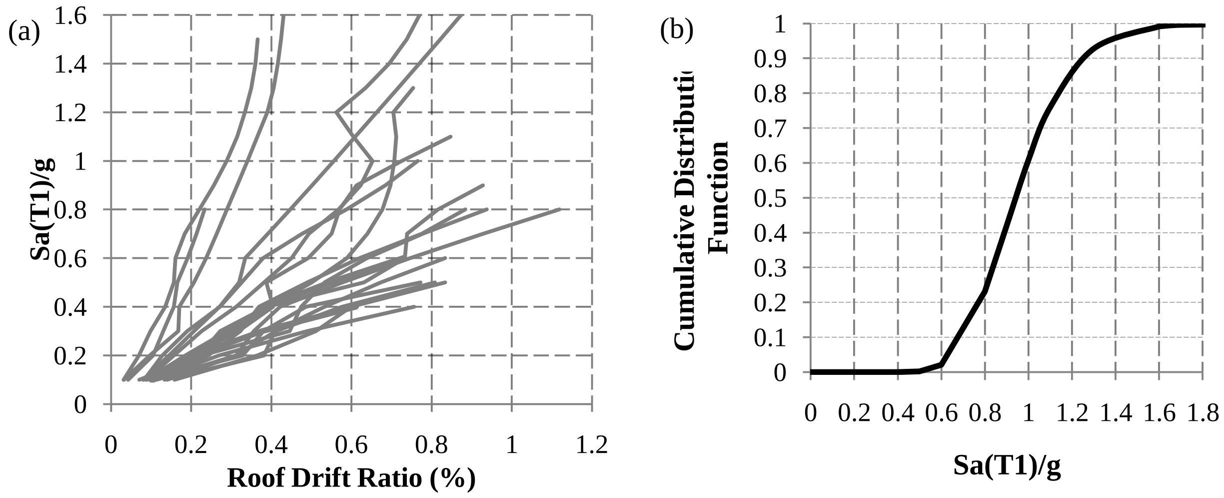

Figure 7 Collapse capacity assessment of the case study building via IDA (a) IDA curves (b) Collapse fragility curve

Although application of the fiber model used in this paper is a step towards improved structural behaviour simulation compared to lumped plasticity models, many aspects of structural modelling are yet to be improved. Collapse due to brittle shear in reinforced concrete components cannot be captured in the model. Furthermore, ideally the stress carried by confined concrete gradually decreases to a residual stress after the peak point while in the model used herein, the stress abruptly falls to zero after crushing. The steel model used in this study also does not simulate fracture of steel bars at the ultimate strain. However, it was shown that for the example building, compression concrete crushing occurs prior to steel failure, and hence ignoring the rupture of the bar at the ultimate strain does not add further limitation on the model. Slab effects on the capacity of structural components, effect of non-structural components in strength and stiffness of the frame, as well as soil–structure interaction effects are also among the parameters that were neglected in this study.

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