IMPACT OF MATERIAL CYCLIC DEGRADATION ON NONLINEAR DYNAMIC RESPONSE OF RC BRIDGE PIERS

M.M. Kashani[1], C. Málaga-Chuquitaype [2], S. Yang[3], N.A. Alexander[4], A.J. Crewe[5]

[1] *Lecturer, University of Bristol, mehdi.kashani@bristol.ac.uk*
[2] *Lecturer, Imperial College London, c.malaga@imperial.ac.uk*
[3] *Master's Student, Imperial College London, shijia.yang14@imperial.ac.uk*
[4] *Senior Lecturer, University of Bristol, nick.alexander@bristol.ac.uk*
[5] *Reader, University of Bristol, a.j.crewe@bristol.ac.uk*

Abstract

The current performance-based seismic design philosophy of reinforced concrete (RC) structures relies on the proper detailing of plastic hinge regions where most of the inelastic deformations are expected to occur. The inelastic cyclic deformation in plastic hinge regions results in a significant tension and compression strain reversals. Unlike buildings where plastic hinges are designed to occur in beams, due to the nature of the structural system of bridges the plastic hinges are forced to occur in piers. As a result, they should be able to accommodate a significant inelastic deformation due to earthquake loading. One of the most common failure modes of RC bridge piers that has been observed in real earthquakes and experimental testing is the buckling of vertical reinforcement. This is then followed by either confined concrete crushing in compression and/or fracture of reinforcement in tension due to low-cycle high amplitude fatigue degradation. Earlier research resulted in the development of a novel nonlinear material model for reinforcing bars that accounts for the effect of inelastic buckling and low-cycle fatigue degradation of reinforcing bars. This paper discusses a new modelling technique that is able to predict the nonlinear cyclic response of bridge RC bridge piers up to complete collapse. This model has been validated and calibrated against experimental data. Three groups of ground motions are selected to represent the far field (FF), near field without pulse (NFWP) and near field pulse-like (NFPL) ground motions with a range of PGAs and durations. The response spectra of all ground motions are matched to the mean response spectrum of the far field ground motions group. Using the selected ground motions several incremental dynamic analyses (IDA) of a representative RC bridge piers with various fundamental periods (various heights) are conducted. Finally a comparison between the response of the structure using the new material model (accounting for both buckling and low-cycle fatigue) and the conventional material model for reinforcing steel (without buckling and any degradation) are made.

Keywords: Incremental dynamic analysis, low-cycle fatigue, ground-motion duration, nonlinear analysis, inelastic buckling

1. Introduction

Current state-of-the-practice and modern seismic design codes [1,2] rely on the proper detailing of the plastic hinge regions where most of the inelastic deformations are expected to occur. Therefore, the structural performance is greatly influenced by the structural details (e.g. sufficient confinement for concrete) and material performance (e.g. ductility of reinforcing steel). Furthermore, given that earthquakes are an extreme cyclic dynamic loading case, they result in significant tension and compression strain reversals at critical cross section in plastic hinge regions. This subsequently leads to low-cycle high-amplitude fatigue degradation [3-5]. Bridge piers are the most important and critical elements in bridges due to the nature of their structural system. Unlike building structures, where plastic hinges typically initiate in beams [6], in bridges plastic hinges only occur in piers with the deck remaining elastic. Therefore, bridge piers must sustain very large inelastic deformations without collapsing during seismic events to dissipate energy. The combined effects of the large lateral deformation and axial force in bridge piers can induce inelastic buckling of vertical reinforcement, which subsequently results in crushing of core confined concrete [4,7]. [8-10] reported that inelastic buckling of reinforcing bars has a significant impact on the reduction of their low-cycle high-amplitude fatigue. Furthermore, they found that the low-cycle fatigue failure of reinforcement has a significant load-path dependency. Therefore, it is very important to account for the influence of material degradation under seismic loading in performance prediction of reinforced concrete (RC) bridge piers using nonlinear dynamic analysis.

Furthermore, characterising, by a few salient parameters, the influence of a particular ground-motion time-series, on the nonlinear response of some structural systems is a vexed problem. Engineers and seismologists have employed a range of amplitude, energy, averaged frequency content, duration and envelope shape measures, etc. [11,12] to attempt to capture the most significant factors that govern the nonlinear response of the system. [12] summarised previous studies and reported that the conclusions are greatly influenced by which structural demand parameters are considered. For example, comparing the responses to a far-field ground motion and short near-field pulse-like ground motion may not influence the maximum peak drift on the structure. However, given that the far-field ground motions are normally longer they will affect the accumulated damage on the structure due to low-cycle high-amplitude fatigue degradation of materials [11]. The current seismic design codes and loading protocols for component testing of structural components do not account for the influence of ground-motion characteristics e.g. duration, pulse effect. In recent years, new standards such as [13] provide some guidelines to account for the ground-motion duration qualitatively. However, they do not have a well-defined framework for quantifying and accounting for the ground-motion characteristics.

A recent study conducted by [14] proposes a methodology to quantify the effect of ground-motion duration on the probability of structural collapse. In order to isolate the effect of duration from other ground-motion characteristics, such as amplitude and frequency content, they used spectrally equivalent, long and short duration record sets, with unmodified spectral content. Cyclic deterioration of the structural component is implicitly considered via a lumped-plasticity model for a single bridge pier using the Ibarra et al. [15] model. It should be noted that prior to [14], none of the earlier numerical models accounted for the effect of cyclic deterioration in the nonlinear structural models. However, the Ibarra et al. [15] model that is employed in [14] study, does not account for cross section geometry and axial force-bending moment interaction of RC sections and components. This model has to be calibrated for a specific cross section geometry, reinforcement arrangement and axial force ratio. Therefore, it is not able to capture the influence of the loading spatiotemporal history on the failure mode and cannot predict the multiple failure modes.

Accordingly, this paper reports the summary of a novel approach to quantify the impact of the ground-motion time-series profile (near/far field, with/without pulses) on the nonlinear dynamic response of RC bridge piers. The new algorithm (known as RVSA) developed by Alexander et al. [16] is employed to generate a set artificial ground motions of equivalent spectral response. The ground-motion seeds are selected from the suggested far-field (FF), near-field without pulse (NFWP) and near-field pulse like (NFPL) ground motions in FEMA P695 [17]. These ground motions have differences in amplitude, duration and power spectral content, i.e. differences in stationary and non-stationary components. Then, using the RVSA they are all matched to a target response spectrum (without qualitatively changing the non-stationary ground-motion characteristics, i.e. envelope and pulses) to be used in nonlinear dynamic analyses. This will isolate the influence of ground-motion envelope and pulses (non-stationary effects) from ground-motion response spectral characteristics (stationary effects).

Finally, using the spectrally matched and unmatched ground motions, a series of Incremental Dynamic Analyses (IDAs) are conducted on three prototype circular RC bridge pries (taken from Lehman and Moehle [7] test units) with different heights (representing various elastic fundamental periods, T1). A comparison is made between the nonlinear dynamic response of the proposed bridge piers obtained with the advanced model accounting for inelastic buckling and low-cycle fatigue of reinforcement (developed by Kahsani et al. [18]) and the conventional model without consideration of bar buckling and degradation.

2. Experimentally Tested Bridge Pier Models

Three bridge piers varied in heights are employed in this study. The details of the proposed bride piers are shown in Table 1. These columns are taken from experimental test units reported in [7]. The same ID as used in the experiment is employed herein to identify these columns. The test specimens used in this study are units 415, 815 and 1015. A schematic illustration of these columns is shown in Fig. 1 and geometrical dimensions and reinforcement details are summarised in the Table 1. In Table 1, L/D is the ratio between column length (L) and column diameter (D), ρ_l is the longitudinal reinforcement ratio as ratio of the total cross section area of RC column, ρ_h is the volumetric ratio of the transverse reinforcement, $P/(A_g f_c)$ is axial force ratio where P is axial force, A_g is the gross area of column cross section, and f_c is the concrete compressive strength. Further detailed information about the experimental tests and material properties are available in [7]. The digital force-displacement data of these experiments are available at the UW-PEER column database [19].
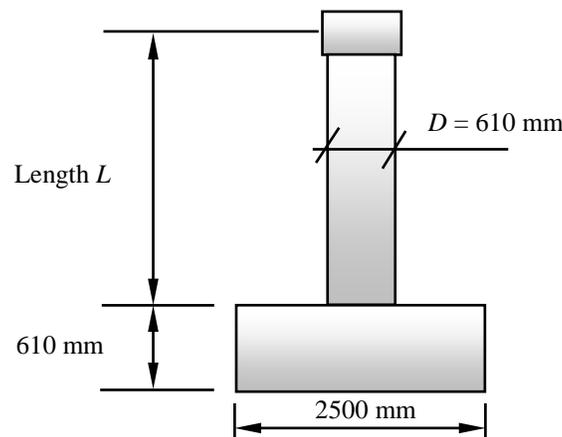
Fig. 1 Schematic view of the bridge piers tested by[7]

Table 1 Details of experimental test units as reported in [7] (units in mm)

Column ID	Length L	L/D	Vertical Bar Dia.	No. of Vertical Bars	Horz. Bar Dia.	Horz. Bar Spacing	$P/(A_g f_c)$
415	2438.4	4	16	22	6.5	32	0.07
815	4876.8	8	16	22	6.5	32	0.07
1015	6096	10	16	22	6.5	32	0.07

3. Finite Element Model Description

There are several methods available to model the nonlinear behaviour of RC structures. One of the most recent finite element techniques that is very popular among the earthquake engineering community is the fibre-based section discretisation technique. In this technique the RC component is modelled using a beam-column element. Then, the member cross section is discretised into a number of steel and concrete fibres at selected integration points. The material nonlinearity is represented through a uniaxial constitutive material model of steel (tension and compression) and concrete (confined core concrete and unconfined cover concrete). Therefore, the accuracy of the nonlinear model's response of RC components and structures is greatly influenced by the accuracy of the uniaxial material model. Accordingly, several researchers have developed material models for concrete and reinforcing steel that can be used in nonlinear fibre beam-column element formulations. More recently, [8] developed a new phenomenological uniaxial material model that has been implemented in OpenSees. [18] also developed an advanced modelling technique in OpenSees [20] that employs their new uniaxial material model to simulate the nonlinear behaviour of circular RC bridge piers. This research employs the same model that has been extensively validated against a variety of experimental datasets. In this research the effect of geometrical nonlinearity and large deformation is also considered in addition to the material nonlinearity.

3.1. Description of nonlinear uniaxial material models

3.1.1 Uniaxial material models for concrete

Concrete04 available in OpenSees is used to model the confined and unconfined concrete. The confined concrete material is located in the core of the RC column in which concrete is confined by horizontal tie reinforcement. Unconfined concrete material represents the cover concrete. The *Concrete04* employs the Popovics curve for the compression envelope and the Karsan-Jirsa model to determine the slope of the curve for unloading and reloading in compression [20]. For tensile loading, an exponential curve is used to define the envelope to the stress-strain curve. Further details are available in [20]. The parameters proposed by Mander et al. [21] are used to model the effect of confinement on concrete in compression. The maximum compressive stress of the concrete and the strain at the maximum compressive strain (confined concrete strain at maximum stress) can be calculated using the equations developed in [21]. It should be noted that the maximum crushing strain of the confined concrete is limited by the fracture of the first horizontal tie/spiral reinforcement. In this research the empirical model developed by [21] is used to define the confined concrete crushing strain in *Concerete04*.

3.1.2 Uniaxial material models for reinforcing steel

Two types of material models have been considered in the analyses. The first model is the conventional Giuffre-Menegotto-Pinto (GMP) model [22]. This model was modified by [23] and later implemented in OpenSees as *Steel02*. The *Steel02* accounts for the Bauschinger effect, but does not account for cyclic strength and stiffness degradation due to bar buckling and fatigue. The second model is the phenomenological uniaxial model developed by [8]. The generic Hysteretic material model in OpenSees is used for its implementation. The generic Fatigue wrapper developed by [24] is then wrapped to the Hysteretic material model to simulate the low-cycle fatigue failure of vertical reinforcing bars. Therefore, this combined model accounts for the influence of inelastic buckling and low-cycle fatigue degradation. Through a comprehensive parametric study the 'optimum' pinch parameters of the model under cyclic loading are obtained [18]. A detailed discussion of the model development and calibration is out of the scope of this paper and is reported elsewhere [8,18].

3.2 Finite element model verification

Fig. 2 shows a qualitative comparison between OpenSees simulations and the observed experimental results. Fig. 2 shows that the simulation results using *Steel02* material model can predict the nonlinear cyclic response accurately up to the maximum strength. However, as the lateral deformation of columns increases, severe strength degradation is observed in the experiment that cannot be simulated by *Steel02*. The simulation results using the buckling and fatigue model show that the failure mode of these columns is initiated by inelastic buckling of vertical bars. As the damage progresses after buckling of vertical bars the core concrete starts to crush. This is followed by fracture of vertical bars in tension due to low-cycle high amplitude fatigue. These observations using the simulation results are in good agreement with the experimental failure modes reported in

[7]. Therefore, it is evident that the proposed nonlinear fibre beam-column model is able to reliably simulate the nonlinear response of these bridge piers up to complete collapse.

(a)

(b)

(c)
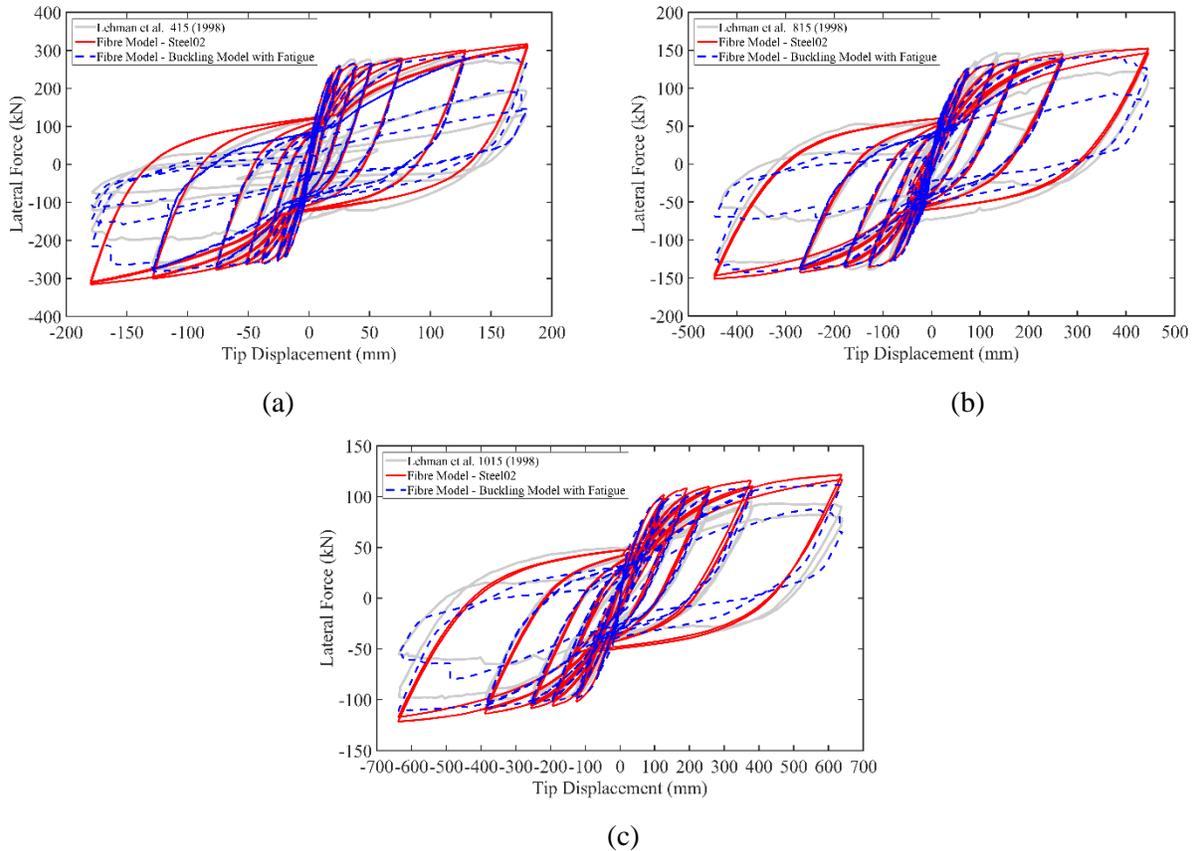
Fig. 2 Comparison of the computed response using OpenSees and observed experimental response: (a) test unit 415 (b) test unit 815 and (c) test unit 1015

4. Ground motion selection and matching

In this study three types of ground motions were considered. This includes Far-Field (FF), Near-Field without Pulse (NFWP) and Near-Field Pulse-Like (NFPL) record sets. Ground motions close to a ruptured fault can be significantly different than those further away from the seismic source. The near-field zone is typically assumed to be within a distance of about 20-60 km from a ruptured fault. Structures with medium to short natural vibration period are more vulnerable to near-field type ground motions. A pulse-like ground motion is considered to be a record with a short-duration pulse that occurs early in the velocity time history and has large amplitude [25]. One cause of these velocity pulses is forward-directivity effects in the near-fault region. Forward directivity results when the fault rupture propagates toward the site at a velocity nearly equal to the propagation velocity of the shear waves and the direction of fault slip is aligned with the site. This causes the wave front to arrive as a single large pulse [26]. Therefore, depending on the pulse period, structures with longer period may also be vulnerable to pulse-like ground motions.

4.1   Ground-motion selection

FEMA P695 [17] provides a list of ground motions with different characteristics i.e. FF, NFPL and NFWP. The recorded acceleration series of these ground motions are available in the PEER-NGA West database [27], which is employed in this research. The FEMA P695 recommends a set of 22 FF records with average moment magnitude of Mw7.0. Each record has two horizontal and one vertical component. They are taken from 14

historical events which happened between 1971 and 1999. Eight of them occurred in California and 6 of them are taken from different places around the world. 16 of these records are related to stiff soil sites and 6 of them are related to very stiff soil sites. The Peak Ground Acceleration (PGA) of these records varies from 0.18g to 0.58g with a mean value of 0.4g. Their Peak Ground Velocity (PGV) varies from 0.36m/s to 0.42m/s with a mean value of 0.42m/s. The FEMA P695 recommends a total of 28 NFPL and NFWP ground-motion records with 56 components that are all available in the PEER-NGA West database. 14 of these ground motions are NFPL and 14 records are NFWP. These ground motions have an average magnitude of Mw7.0 and are taken from 14 events that happened from 1976 to 2002. 7 of these ground motions are recoded in the United States and 5 others come from other countries around the world. 11 ground-motion stations were located in stiff soil sites, 15 of them in very stiff soil sites and the rest correspond to rock sites. Values of their PGAs vary from 0.22g to 1.43g with a mean value of 0.6g. The PGVs of these ground motions vary between 0.30m/s and 0.167m/s with a mean value of 0.84m/s. The elastic response spectra of all ground motions as well as the mean response spectrum of each group are shown in Fig. 3.
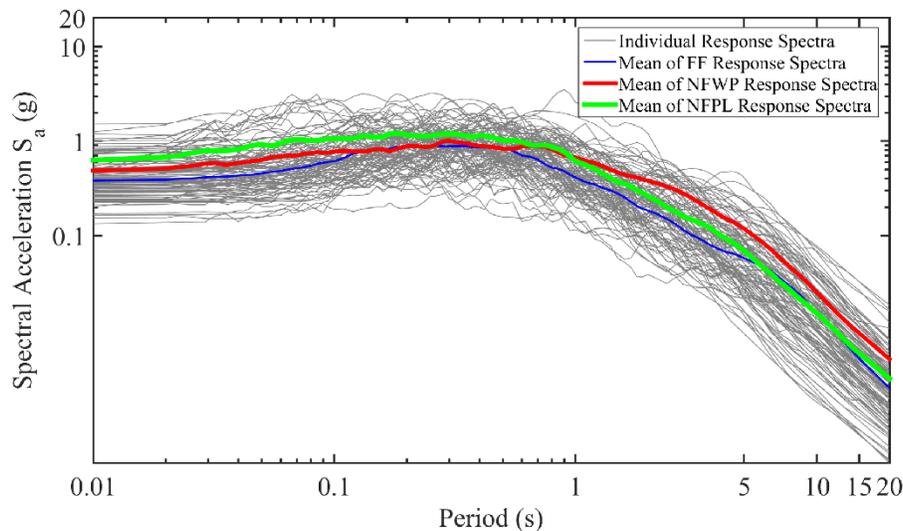
Fig. 3 Elastic response spectra of FEMA P695 recommended ground motions

4.2 Matching selected ground motions to the target response spectra

In order to determine the effect of the acceleration time series non-stationary characteristics (i.e. envelope and timing of pulses) the influence of response spectral content (estimated stationary content) is removed by spectrally matching all records to a common response spectrum. Thus, we ask whether there is any statistical difference between the response of the bridge piers being considered to near/far field and pulse/non-pulse like records when the influence of response spectral differences (stationary content) is removed.

To this end, the Reweighted Volterra Series Algorithm (RVSA) proposed by [16] is employed. The spectral matching (using RVSA) is not sensitive to selection of ground motions which means it provides a well matched time-series from any seed time series. This spectral matching process keeps the non-stationary characteristics (i.e. the general envelope and timing of main pulses) of the seed record largely unchanged but it matches the target response spectrum. The Reweighted Volterra Series Algorithm (RVSA) re-expresses the seed time series as a discretised Volterra series. A multilevel wavelet decomposition, using the stationary wavelet transform, is employed to estimate the first order Volterra kernel. Higher order Volterra kernels are estimated using multinomial mixing of first order kernel terms. The weighting of each term with each Volterra kernel is obtained using Levenberg-Marquardt nonlinear optimisation. As the number of kernels employed increases the dimensionality of the nonlinear optimisation problem exhibits near-exponential growth. Thus, it becomes computationally expensive to use too many high order kernels however a large degree of convergence is

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normally achieve with two or three kernels. Further details of this procedure are available in Alexander et al. [16].

5 records per ground-motion set are selected from the recommended ground motions in FEMA P695. The records are taken from PEER-NGA West database. Thus, the response spectra of these three ground motion groups are matched to the mean response spectrum of all FF ground-motions recommended in FEMA P695. The matched response spectra of these ground-motions are shown in Fig. 4.
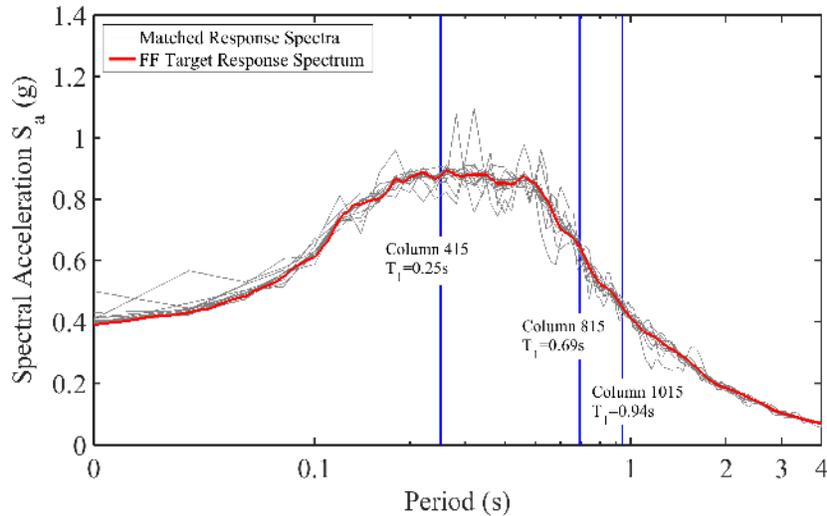
Fig. 4 Matched response spectra of selected ground motions

5. Incremental dynamic analysis (IDA), results and discussion

One of the most popular methods for collapse assessment of RC structures is incremental dynamic analysis (IDA) [28]. In this method, the performance of a structure under a set of ground motions of increasing intensity can be investigated. In this research, a series of IDAs are performed, in order to explore the influence of material degradation (buckling and fatigue) on the nonlinear dynamic response of RC bridge piers. The IDAs are conducted for each group of matched and equivalent unmatched ground motions that were discussed in section 3 of this paper. The intensity of ground motion was characterised herein by the value of its spectral acceleration (Sa) at the fundamental period (T1) of the structures i.e. Sa(T1).

The distribution of peak drifts (mean and standard deviation) is estimated directly from the structural responses. To this end, a log-normal distribution was fitted by the method of moments. Collapse is defined to have occurred if the drift ratio exceeds 15%. In order to account for this, the probability of collapse is first estimated and then the non-collapse responses are employed for distribution fitting.

5.1   Influence of material degradation on nonlinear response

Fig. 5 shows the analyses results of the Case I ground motions. In Case I, all of the ground motions are matched to the mean response spectrum of the FF ground motions. Fig. 5(a) shows that Column 415, including the effect of buckling and fatigue, would fail at smaller drifts (at about 7% drift) than that of the model without buckling and fatigue. Column 815 (Fig. 5(b)) also shows similar behaviour to 415. However, Fig. 5(c) shows that the response of Column 1015 using the model including buckling and fatigue and the model without buckling and fatigue are almost identical. This indicates that material degradation due buckling and fatigue would not have a significant effect in the behaviour of Model 1015.

Column 415

(a)

Column 815

(b)

Column 1015

(c)
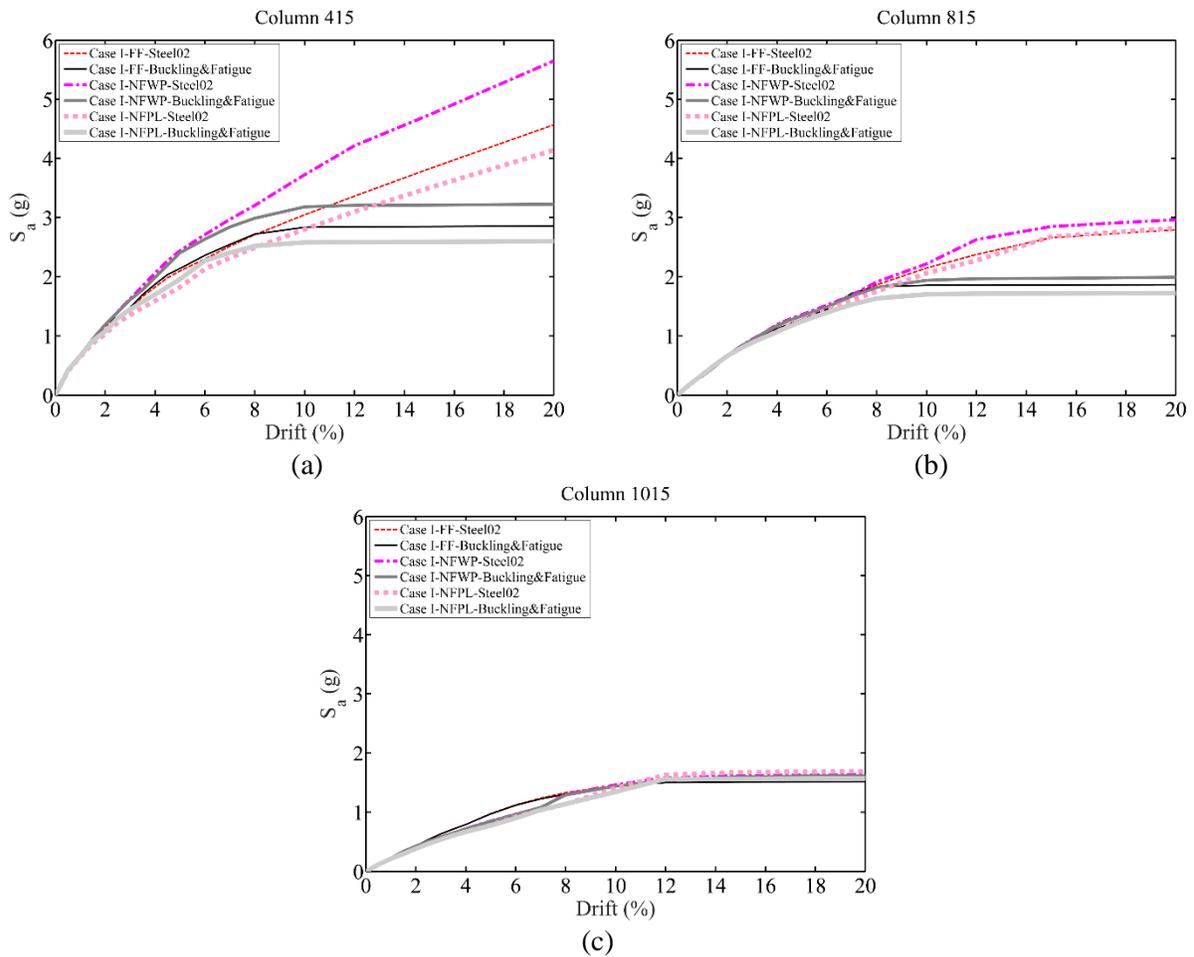
Fig. 5 Mean response of the RC columns under matched ground motions: (a) Column 415, (b) Column 815 and (c) Column 1015

A comparison of Figs. 5(a), (b) and (c), shows that the effect of buckling and fatigue on nonlinear dynamic response of the structure become less significant from Model 415 to Model 1015. The impact of considering buckling and fatigue in capacity reduction of each column is shown in Fig 7. The ratios of the mean computed responses of the model including buckling and fatigue and the model without buckling and fatigue, for each column, under Case I ground motions are plotted in Fig. 6.
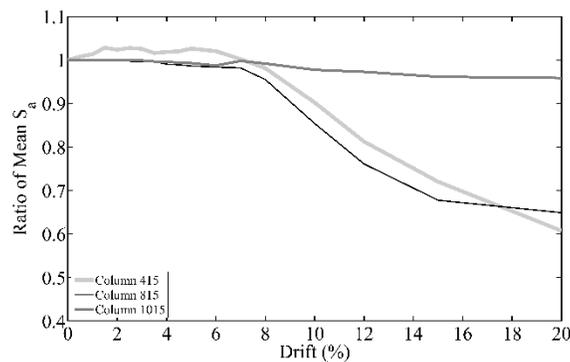
Fig. 6 Ratio of mean response (mean Sa) of the model including buckling and fatigue to the model without buckling and fatigue (Case I)

5. Conclusions

A series of IDAs are conducted on three prototype RC bridge piers with various heights. The ground motion records were selected from suggested ground motions in FEMA P695. Three types of ground motions such as FF, NFWP and NFPL are considered. To investigate the impact of the non-stationary components associated with different ground-motion types (e.g. near-field and pulse effect compared to far field effect) and exclude it from other ground-motion frequency-related characteristics, a response spectrum matching technique is employed. To this end, all of the ground motions are matched to the mean response spectrum of FF ground-motions. The main conclusions of this study are summarised as follows:

1. It was found that considering inelastic buckling and low-cycle fatigue resulted in a reduction of the peak response (maximum strength) of about 40% in columns 415 (L/D = 4) and 815 (L/D = 8). However, it didn't have a significant influence on the response of Column 1015 (L/D = 10).

2. The influence of inelastic buckling and degradation on the response of the columns is reduced by increasing the height of the columns. Similar behaviour is observed in static cyclic testing of these columns. This is because when the column is taller, a small normalised lateral deformation at top of the column results in a larger rotation at the base. As a result the failure mode is generally governed by fracture of the vertical reinforcement in tension. In other words, the taller the column the more likely it fails in tension before it experiences any significant cyclic degradation due to inelastic buckling and low-cycle high-amplitude fatigue.

3. The outcome of this study shows that the material degradation due to inelastic buckling and low-cycle fatigue of reinforcement must be considered in seismic performance, design, analysis and assessment of existing and new structures. Therefore, the response spectrum analysis and/or capacity spectrum techniques that rely on the elastic period and static inelastic response (pushover curves) of structures are not accurate enough for performance prediction. They can be used as a simplified starting point, however, more comprehensive analyses are required depending on the complexity of the structure, ground-motion characteristics and construction materials of the structure.

4. The method proposed and used in this paper provides a comprehensive platform to be used by earthquake engineering community (researchers and practitioners) in future research. The nonlinear structural model is readily available in OpenSees and the response spectrum matching software is available for free download at https://sites.google.com/site/volterramatch/.

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