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A comparative study on nonlinear models for performance-based earthquake engineering

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ABSTRACT

Performance-based earthquake engineering requires a large number of nonlinear dynamic analyses to statistically assess the performance of frame structures. The complexity and high computational demand of such procedures, however, has hindered its use in practice. The objective of this study is to evaluate the performance of three numerical models with varying computational demand levels. Two nonlinear models with different complexities and one linear model with a concentrated plasticity approach were used to evaluate a reinforced concrete frame. The accuracy of the calculated responses was assessed using the experimental results. A total number of 126 dynamic analyses were performed to derive fragility curves. The nonlinear models calculated significantly more accurate structural responses than the more-commonly used plastic-hinge model. The model preparation and result acquisition times were found to comprise a significant portion of the total computational demand of each model. An overview of the performance-based modeling processes and the critical points for minimizing the computational demand while retaining the calculation accuracy are also presented.

1. Introduction

Performance-based earthquake engineering (PBEE) makes use of the nonlinear structural analysis (NLA) methods to accurately predict the inelastic response that most buildings undergo during seismic excitation. Amongst different NLA methods, the nonlinear dynamic analysis (NLDA) methods, also known as time-history analysis, provide the most realistic simulation of structural behavior [1–4]. Multiple NLDAs are required to assess (or design) a structure using PBEE; however, the NLDA methods are complex and computationally-intensive, which significantly limits their applicability in practical situations.

Previous studies have either focused on proposing simplified analysis procedures [4–9] to substitute the need for the NLDA methods or evaluate the influence of local element assumptions and modeling approaches on the overall structural response [10–12]. There is still a lack of studies that investigate the structural response reliability when a structural system is numerically analyzed with different modeling techniques. The objective of this research is to study various numerical modeling techniques with different complexity levels and evaluate their simulation accuracy and computational demand. For this objective, a PBEE structural assessment of a previously-tested RC frame is conducted using three modeling approaches. The calculated structural risk to a set of performance limits is evaluated by means of fragility curves.

2. Performance-based earthquake engineering

A summary of the PBEE structural assessment is presented herein to illustrate the methodology used in this paper [13,14]. First, the building location, importance, and soil condition are used to determine the earthquake hazard level and the response spectrum of the structure as per the applicable building code. Structural analysis is then conducted using a numerical model subjected to a series of ground motion (GM) acceleration histories that match the response spectrum. The performance is evaluated based on the calculated responses and the structural risk is expressed by means of fragility (or vulnerability) curves, which indicate the probability of the structure to exceed a certain damage state (i.e., damage measures or performance levels) based on the engineering demand parameters (EDP) (e.g., story drift, floor accelerations, or velocities) calculated by the structural analysis. A loss analysis is finally conducted, based on the previously calculated probability of exceedance, to quantify the financial, downtime, casualty, or other types of loses.

3. Hazard determination

In this study, the structure considered is in Portland, Oregon, USA, and constructed over ‘type D’ soil, which is the standard soil type in ASCE 7 [15] when no sufficient detail is provided. The design response

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Table 1
Selected ground motion characteristics.

ID	Earthquake name	Year	Station name	Mag.	Epicenter distance, km	Scale factor
1	Imperial Valley-02	1940	El Centro Array #9	6.95	12.98	1.5
2	Imperial Valley-06	1979	Agrarias	6.53	2.62	2.6
3	Victoria, Mexico	1980	Cerro Prieto	6.33	33.73	1.2
4	Superstition Hills-02	1987	El Centro Imp. Co. Cent	6.54	35.83	1.6
5	Landers	1992	Desert Hot Springs	7.28	27.32	2.7
6	Erzican, Turkey	1992	Erzincan	6.69	8.97	1.5
7	Parkfield-02, CA	2004	Parkfield - UPSAR 13	6.00	12.59	2.6

spectrum was calculated based on the NEHRP [16] provisions.

Seven acceleration histories were considered to meet the minimum requirements of the NEHRP [16] provisions. The ground motion characteristics included: 'strike-slip' fault type, less than 50 km to the epicenter, and Richter magnitude between 6 and 8 (see Table 1). Time-histories were obtained from the Pacific Earthquake Engineering Research (PEER) online NGA-West2 database [17].

The selected ground motions were scaled such that the average follows the requirements of NEHRP [16], with the result shown in Fig. 1. The one-third scale frame to be examined exhibited a natural period of 0.303 s, which corresponds to a full-scale period of 0.525 s.

4. Structural analysis

A structure designed based on pre-1970 s building codes was chosen for assessment using the PBEE methods due to their seismically-deficient details. The frame examined was a one-third scale, three-story, three-bay planar structure designed by Ghannoum and Moehle [18] to develop a flexure-shear-critical failure mechanism (i.e., the columns yield in flexure prior to a shear failure). Two of the columns were constructed with widely-spaced shear reinforcement (denoted as non-ductile columns), while the other two columns were designed to fulfill ACI 318-08 specifications (denoted as ductile columns). Ghannoum and Moehle [18] indicated that the mixture of older-type columns and ductile columns is not completely representative of typical 1970s construction. It was introduced in the test frame so that collapse of the frame due to the failure of the older-type columns would be slowed by the ductile columns and the dynamic failure mechanism could be more closely monitored. A strong beam-weak column mechanism was included, and the beam-column joints were designed in accordance with ACI 318-08 to avoid any joint failure prior to a column failure. Each beam carried 26.68-kN lead weight packets distributed over two points located approx. 0.4 m from the face of each column. A sketch of the frame is shown in Fig. 2.

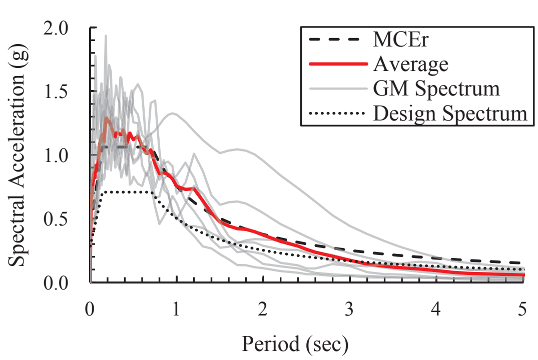

Fig. 1. Spectral response.

The frame was subjected to four shake table tests using the March 3, 1985, Chile Earthquake (Llolleo Station, Component 100); namely, half-yield (HY), and dynamic tests 1, 2, and 3 (DT1, DT2, and DT3). Table 2 lists the ground motion scale factors and the response of the frame in each test [18].

5. Numerical modeling

In this study, three numerical models were created. A full nonlinear model that employs distributed-plasticity fiber-based elements, called Nonlinear Fiber-Based (NLFB) model; a simplified nonlinear model with fewer and longer flexure-only elements with combined shear-hinges, called Nonlinear Fiber-Based Shear Hinge (NLFBSH) model; and a fully-elastic model with concentrated flexure, axial, and shear-hinges, called Elastic with Concentrated Plasticity Hinges (ECPH) model. All models used two-dimensional beam-column elements due to their computational efficiency and analytical accuracy.

5.1. Nonlinear fiber-based model (NLFB)

The NLFB model employed the frame element developed by Guner and Vecchio [19]. This element performs interrelated global and sectional analyses, where the internal forces calculated by the former are used to perform the latter. It is based on the Modified Compression Field Theory (MCFT) [20], which allows the element to account for the coupled flexure, axial and shear effects. Additionally, the MCFT uses the average and local strains and stresses of the concrete and reinforcement, and the widths and orientations of cracks throughout the load-deformation response of the element. Shear strains are calculated using a parabolic strain distribution [19]. The element employs a smeared, rotating crack approach based on a total load, secant stiffness formulation. The triaxial concrete core confinement is inherently accounted for through the use of in- and out-of-plane reinforcement ratios. In addition, it incorporates several second-order material behaviors that are specific to reinforced concrete structures, as listed in Table 3 [21].

The structure was modeled using the computer program VecTor 5 [22,23]. The structural analysis package also incorporates graphical pre- and post-processor programs. FormWorks Plus [24,25] is a graphical pre-processor developed specifically for the VecTor suite of applications to provide better modeling capabilities such as the list of available elements and material models, auto-meshing and auto-substructure features. The post-processor program Janus [26,27] can display the displaced shape of the structure, crack widths, locations and propagation, rebar and concrete stresses and strains, and failure conditions. The post-processor program is a critical component of structural assessment process since they aid analysts to understand the structural behavior, detect modeling mistakes, and effectively compare the calculated responses. Some important capabilities of the computer program VecTor5 are summarized in Table 4.

The concrete uniaxial stress-strain response was modeled using the Popovics and Modified Park-Kent models for the pre- and post-peak responses [21]. The steel reinforcement stress-strain response is composed of three parts: linear-elastic response, yield plateau, and a nonlinear strain-hardening phase until rupture in tension, and a buckling response in compression (see Fig. 3). As recommended by [19], each beam and column was divided into elements of about half of its cross-section height (see Fig. 4), and the number of fibers used in all cross-sections was kept at about 30 fibers. The longitudinal reinforcement was discretely modeled while the shear reinforcement was smeared into relevant concrete layers.

The NLFB model incorporated a nonlinear concrete model with plastic offsets proposed by [28]. In this model, the concrete unloads to a plastic offset strain, not to the origin of the stress-strain diagram, following a nonlinear Ramberg-Osgood formulation. The reinforcing steel hysteretic response was based on the Seckin model with Bauschinger