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SEISMIC ASSESSMENT OF THE STEEL MOMENT RESISTING FRAME EQUIPPED WITH FRICTION BEAM-TO-COLUMN JOINTS

A. poursadrollah[1]

[1] Ph.D.
University of Naples Federico II, Department of Civil Engineering, Naples, Italy
arash.poursadrollah@unina.it

Abstract

Enhancing the seismic response of structures with supplementary dissipative devices is an effective design strategy. A possible low-cost and easy to maintain solution to optimize the seismic performance of steel Moment Resisting Frames is inserting friction devices into beam-column joints. The applicability of such a system will be demonstrated by building a real-scale structures in the campus of the University of Salerno. The present paper aims to evaluate the seismic behavior of the prototype building using the framework of performance-based earthquake engineering method at different performance levels. To this end, nonlinear dynamic analyses are carried out on a three-dimensional (3-D) models of the prototype developed in "OpenSees". The results show that this type of structure can be free of damage under earth-quakes corresponding to the design and maximum considerable levels.

Keywords: Seismic assessment, friction device, steel moment resisting frame, OpenSees.

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1 INTRODUCTION

Steel structures are widely used in seismic areas [1-11] since, if properly designed, they can provide large dissipation of earthquake energy exploiting the material ductility [12-14]. In this work, the seismic assessment of steel moment resisting frames, equipped with friction beam-to-column joints developed through FREEDAM project, was investigated. Supplemental energy dissipation devices are widely used to control the structural response to earthquakes and wind forces. Differently from the traditional systems, where the energy is dissipated by the plastic deformation of structural elements [15-20], passive systems, such as friction devices, offer a wide variety of features such as ease of maintenance and are easily replaced in the event of having damage [21-22]. Moreover, such a system can be a potential solution to one of the major criticisms of EC8-compliant steel MRFs which is their large overdesign due to the strict requirements for P-Delta effects and serviceability checks [23]. A formal process for the problem of assessment has been developed by the Pacific Earthquake Engineering Research (PEER) Centre. It involves several stages, such as quantifying seismic ground motion hazard, structural response, damage to the building and contents, and resulting consequences (financial losses, fatalities, and business interruption). Using a conditional probability distribution format, each stage of the process is performed independently, making the framework modular [24-25].

2 NUMERICAL MODELING

The building prototype of this study is a 3-story MRF, whose plan configuration is shown in Figure 1. The building has a first-story height of 3.2 m and a typical story height of 4.2 m. The three-dimensional (3D) model of the building is developed in OpenSees [26]. In order to consider the interaction between moments and axial forces within columns, the columns are force-based beam-column elements with five fiber sections along their height. Similarly, the beams are force-based beam-columns but with aggregated sections since the interaction of axial and bending moment is negligible in beams. The flexural behavior has been obtained through a moment-curvature analysis of the section and is aggregated with the axial behavior of the section into one single section.

It should be noted that there is no interaction between responses in different degrees of freedom (DOFs). The behavior of the Freedam joint is obtained from a detailed model in Abaqus and experimental activity. Subsequently, a model (i.e., Pinching4 Material) in OpenSees is calibrated and assigned to the zerolength positioned between the nodes of columns and beams as shown in Figure 1. Gravity columns and beams are simple elastic

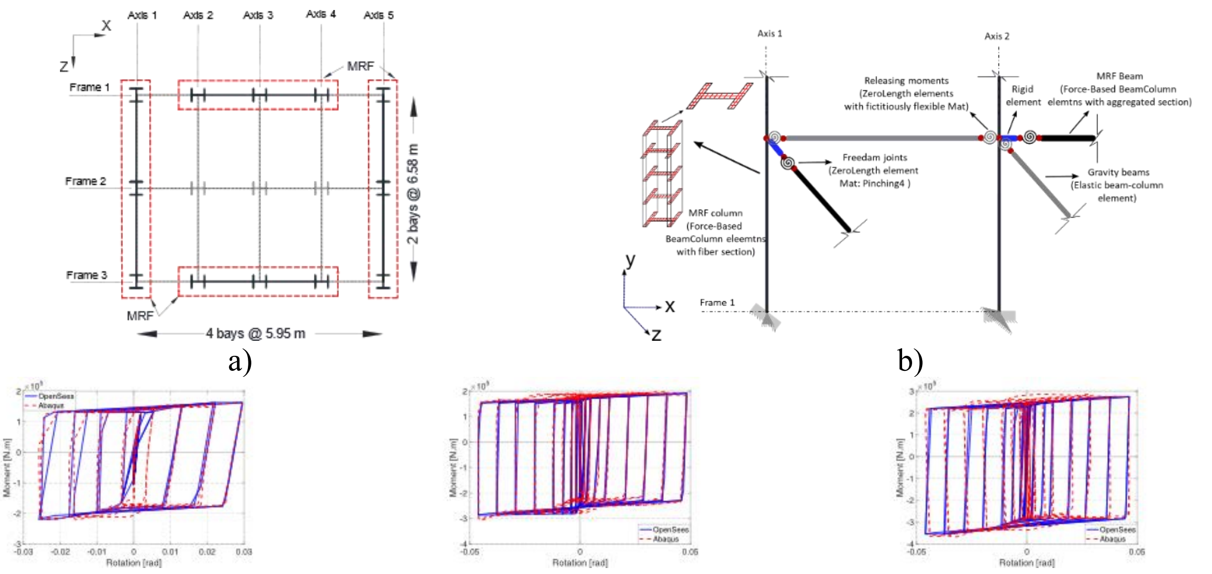


Figure 1 The plan view of the prototype building b) The details of numerical modeling in OpenSees c) Calibration of the Freedam joint hysteresis behavior

beam-columns with their corresponding section properties. In addition, rigid offsets from joints are modeled using stiffened elastic elements. However, since the gravity beams are connected to the column's web their length is computed from the center line of the columns. The 1st-period mode of the building in X and Z directions is 0.91 and 1.00 seconds respectively.

3 PERFORMANCE BASED EARTHQUAKE ASSESSMENT

In this section, the performance of the investigated building is evaluated through the framework of performance-based earthquake engineering method. The typical output of this method is the Mean Annual Frequency (MAF, $l_{b.'s}$) of exceeding various levels of performance levels. Using Equation 1, the MAF for a given structural system can be calculated by integrating its fragility curve over the corresponding hazard curve, where, $P_{L.S}|IM$ is the probability of exceeding a limit state at a given intensity measure (IM) and $di_m(IM)/d(IM)$ is the slope of the seismic hazard curve given this IM.

$$\lambda_{L.S} = \int_0^\infty (P_{L.S} \mid IM) \left| \frac{d\lambda_{IM}(IM)}{d(IM)} \right| \tag{1}$$

To compute the MAF of a limit state, two components are needed: the input ground motions and a structural analysis method to establish a probabilistic model for the structural response as a function of the intensity measure.
As the IM, the spectral acceleration of the arbitrary component of the ground motion at the dominant period of the structure ($S_a(T_{z1})$,5%) is selected. Choosing the structural response (i.e., Engineering Demand Parameter or EDP) for this space frame is relatively straightforward. Since maximum interstory drifts are generally considered to be a good predictor of damage, the maximum peak SRSS drift ($\theta_{s,max}$, i.e., the maximum over all stories of the peak of the square-root-sum-of-squares sum) of each story's instantaneous drifts in the two principle directions, is chosen.

In this paper, the incremental dynamic analysis (IDA) is used to determine the distribution of IM values that cause a given EDP level to be reached. In this method, a structural model has subjected to one (or more) ground motion records, each scaled to multiple intensity levels, which result in curves of structural response parameterized by intensity levels. More details can be found in [27].

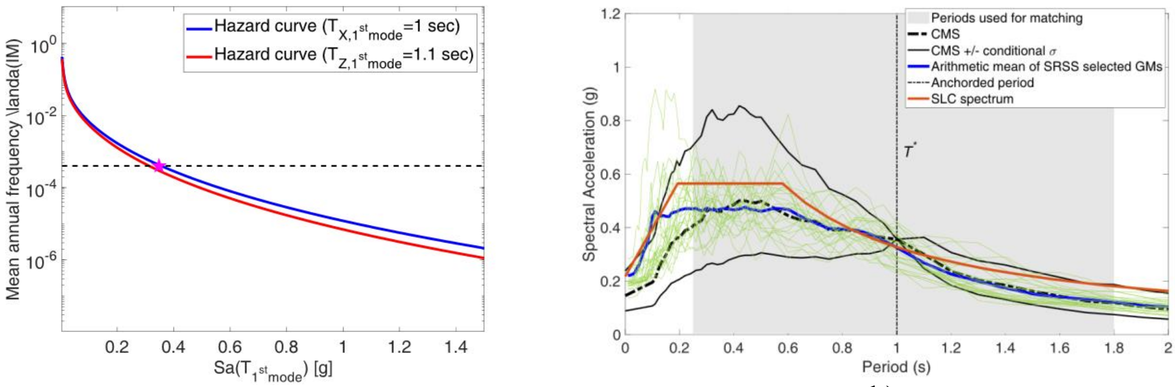
4 GROUND MOTIONS SELECTION

The spectral shape has important effects on structural response; this is especially true when higher mode effects are important or when the building is significantly damaged, causing the effective fundamental period to elongate. Therefore, the input ground motions play an important role in the assessment of a building, particularly at high-intensity levels. In this study, the Conditional Mean Spectrum (CMS) is used as an alternative to the classical code-based record selection. It should be noted that the coordinates of a uniform hazard spectrum are not correlated to each other (i.e., they are simply a collection of individual points). In light of this, it is extremely unlikely to find ground motions with the same spectral amplitudes over its spectrum as the UHS in nature. CMS method anchors the spectrum to spectral acceleration ordinate at a single vibration period T* (i.e. the fundamental period of vibration), and then other spectral accelerations can be calculated by conditioning to the single period. More details can be found in [28]. The CMS and the hazard data are obtained from [29]. A set of 30 ground motion records listed in Table 2 have been selected from the PEER strong ground motion database.

Earthquake name	Year	Mw	Vs-30 (m/s)
San Fernando	1971	6.61	415.13
San Fernando	1971	6.61	389
Irpinia_ Italy-01	1980	6.9	455.93
Corinth_ Greece	1981	6.6	361.4
Coalinga-01	1983	6.36	541.73
Coalinga-01	1983	6.36	441.37
Coalinga-01	1983	6.36	386.19
Coalinga-01	1983	6.36	648.09
Loma Prieta	1989	6.93	415.27
Loma Prieta	1989	6.93	627.59
Loma Prieta	1989	6.93	621.2
Loma Prieta	1989	6.93	425.3
Loma Prieta	1989	6.93	671.77
Northridge-01	1994	6.69	416.58
Northridge-01	1994	6.69	452.15
Northridge-01	1994	6.69	499.31
Northridge-01	1994	6.69	435.64
Northridge-01	1994	6.69	402.16
Chi-Chi_ Taiwan-03	1999	6.2	553.43
Chi-Chi_ Taiwan-03	1999	6.2	573.02
Chi-Chi_ Taiwan-06	1999	6.3	553.43
Chi-Chi_ Taiwan-06	1999	6.3	487.27
Chi-Chi_ Taiwan-06	1999	6.3	454.55
Chuetsu-oki_ Japan	2007	6.8	655.45
Chuetsu-oki_ Japan	2007	6.8	572.37
Chuetsu-oki_ Japan	2007	6.8	425.06
Iwate_ Japan	2008	6.9	555.96
Iwate_ Japan	2008	6.9	593.12
Iwate_ Japan	2008	6.9	561.59
Iwate_ Japan	2008	6.9	436.34

Table 1: Characteristics of the selected ground motion.

Figure 2 shows the response spectrum for selected ground motions that are scaled to match the CMS and the site-specific hazard. The dashed line in Figure 2b is the occurrence of an earthquake with a return period of 2500 years.



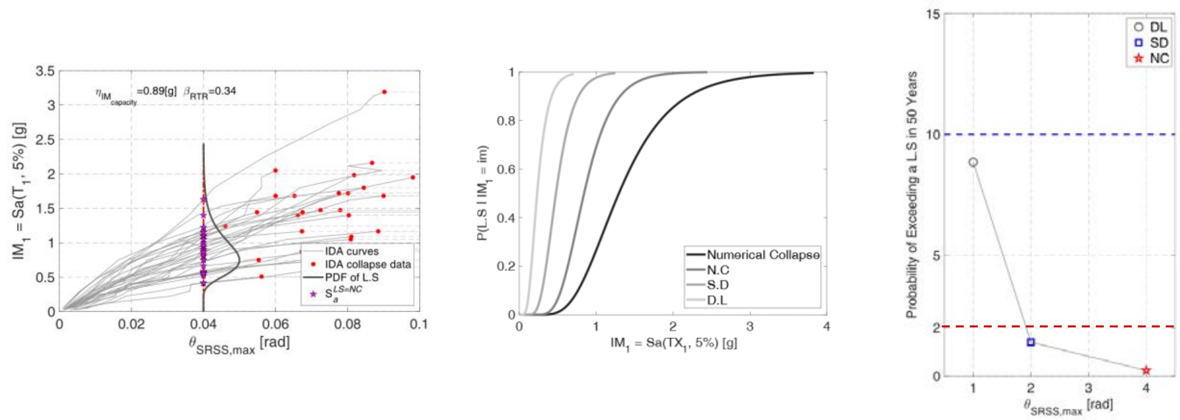
a) b)

Figure 2 a) The site specific hazard b) CMS for (with T*= 1s), and response spectra from ground motions selected to match CMS.

5 RESULTS

Figure 3a shows the IDA curves for selected ground motions along with the fitted lognormal distributions to the collapse points (numerical collapse). The performance of the investigated structure is evaluated at collapse as well as damage limitation (DL), significant damage (SD) and near collapse (NC) limit states based on EC8. These limit states are at fixed interstory drift ratio (theta IDR) and correspond to 1%, 2% and 4% respectively. Therefore, the probability of exceeding a predefined limit state given an occurrence of IM (i.e., Fragility) is obtained by cutting through the IDA curves vertically as shown in Figure 3a. The mean annual frequency of exceeding a limit state, $\lambda_{L.S}$, is computed using Eq. 1. $\lambda_{L.S}$ can be further converted into the probability of exceeding a limit state using the Poisson distribution for the occurrence of an earthquake.

The performance of the structure in each limit state can be compared to a maximum allowable target probability such as the well-known 10% and 2% probability of exceedance in 50 years as shown Figure 3c. It can be observed that the structure satisfies the investigated limit states.



a) b) c)

Figure 3 a) IDA results b) Fragility for various limit states c) performance assessment of the structure.

6 CONCLUSIONS

This study presents a comprehensive analysis of the performance of a prototype building that incorporates freedam joints using a detailed nonlinear 3D finite element (FE) model. The evaluation is performed by means of Incremental Dynamic Analysis (IDA), which enables a thorough investigation of the structure's behavior under various limit states, including significant damage and near collapse.

The results demonstrate that the building exhibits acceptable behavior under the investigated limit states. Specifically, the nonlinearity in the structure is mainly concentrated in the joints, while the rest of the structure remains in an elastic (damage-free) regime. This finding suggests that the freedam joints are effective in dissipating energy and reducing the potential for damage in the structure. Moreover, the probability of damage in the columns is found to be relatively low, with only a 6.7% likelihood of damage occurring under an earthquake corresponding to the NC limit state. This finding highlights the robustness of the structure and the effectiveness of the freedam joints in reducing the potential for damage, which is critical for ensuring the safety and resilience of the building in the face of seismic events.

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