APPROXIMATE METHOD FOR PERFORMANCE-BASED SEISMIC ASSESSMENT OF STEEL MOMENT-RESISTING FRAMES

S-H. Hwang[(1)], D.G. Lignos[(2)]

[(1)] *Graduate Student, McGill University, seong-hoon.hwang@mail.mcgill.ca*
[(2)] *Associate Professor, Swiss Federal Institute of Technology, Lausanne (EPFL), dimitrios.lignos@epfl.ch*

Abstract

A wide range of approximate methods has been historically proposed for performance-based assessment of frame buildings in the aftermath of an earthquake. Most of these methods typically require a detailed analytical model representation of the respective building in order to assess its seismic vulnerability and post-earthquake functionality. This paper proposes an approximate method for estimating story-based engineering demand parameters (*EDP*s) such as peak story drift ratios, peak floor absolute accelerations, and residual story drift ratios in steel frame buildings with steel moment-resisting frames (MRFs). The proposed method is based on concepts from structural health monitoring, which does not require the use of detailed analytical models for structural and non-structural damage diagnosis. The proposed method is able to compute story-based *EDP*s in steel frame buildings with MRFs with reasonable accuracy. Such *EDP*s can facilitate damage assessment/control as well as building-specific seismic loss assessment. The proposed method is utilized to assess the extent of structural damage in an instrumented steel frame building that experienced the 1994 Northridge earthquake.

Keywords: Approximate method, Rapid structural damage assessment, Steel moment-resisting frames, Instrumented buildings

1. Introduction

An important difficulty in current methods of rapid assessment of seismic vulnerability of frame buildings arises from the fact that they typically require detailed engineering inspections. Sophisticated analytical model representations of the respective building are often needed. These may cause long delays in getting back a building to operational stage even in cases that would most likely be classified as safe and functional after the inspection. The consequences of these delays may be detrimental in terms of social and economic costs particularly for infrastructure that is critical for emergency-response operations such as hospitals, schools and governmental buildings.

The performance-based earthquake engineering (PBEE) framework [1-3] utilizes a seismic demand model of structural and nonstructural building components for computing the earthquake-induced economic losses. This model characterizes the relationship between the engineering demand parameters (*EDP*s) and the ground motion intensity in a probabilistic manner. For such purpose, nonlinear building models are subjected to a suite of ground motions and the building *EDP*s are computed based on nonlinear response history analyses (NRHAs) [4]. A number of researchers have proposed probabilistic seismic demand models for building *EDP*s given the seismic intensity of a ground motion [5-12].

Current standards [13, 14] for computing building *EDP*s rely on approximate methods that typically employ equivalent single-degree-of-freedom (SDF) systems (e.g., displacement coefficient method in ASCE/SEI 41-13 [13]) and the nonlinear static procedure [15] in an effort to reduce the computational cost of NRHA. Such methods are not able to estimate important *EDP*s such as the residual story drift ratios and peak floor absolute accelerations (PFAs) that control stakeholder decisions for building demolition as well as repairs in the aftermath of an earthquake. Erochko *et al.* [16] proposed a predictive equation for estimating residual story drift ratios of steel moment-resisting frames (MRFs) and buckling-restrained braced frames (BRBFs). FEMA P-58 [2, 3] provides a simplified procedure for probabilistic seismic demand analysis for low- and mid-rise buildings with moderate inelastic demands (e.g., story drifts ratios are limited to 4% radians; story drift ratios should not exceed 4 times the corresponding yield drift ratio without excessive component strength and stiffness degradation). More recently, Ruiz-García and Chora [17] proposed the coefficient method for estimating residual story drift demands in multi-story steel frame buildings through regression analysis. The aforementioned methods require the explicit use of nonlinear building models. Therefore, detailed information of the building geometry and material properties is necessary in this case. Such models require an appreciable time investment for their further validation. In that sense, nonmodel-based approaches based on principles of structural health monitoring (SHM) could be a valuable alternative [18, 19]. For instance, Noh *et al.* [20, 21] proposed *EDP* indicators for nonmodel-based seismic damage assessment of steel frame buildings as well as reinforced concrete bridge piers by observing the changes in wavelet energies of the first mode natural frequency of the building/bridge pier undamaged state over time.

This paper presents an approximate method for assessing the seismic vulnerability of steel frame buildings with MRFs. The proposed method is nonmodel-based and it utilizes a wavelet-based damage sensitive feature (*DSF*) as discussed in [20, 21] in order to estimate story-based *EDPs* in steel frame buildings with MRFs at various levels of seismic intensity. The proposed method only uses basic building information such as its total height for *EDP* computations. The efficiency of the proposed method in predicting story-based *EDP*s is compared with that of the FEMA P-58 approximate method [2, 3]. An instrumented steel building that experienced the 1994 Northridge earthquake is also used as a case study to demonstrate the potential use of the proposed method in predicting story-based building *EDP*s for structural and non-structural damage control of instrumented steel frame buildings.

2. Damage sensitive features and validation with large-scale data

In order to develop an approximate method for rapid earthquake assessment of steel frame buildings with MRFs, a nonmodel-based approach is employed based on SHM concepts. In particular, wavelet-based *DSF*s are utilized as discussed in [20, 21]. The wavelet-based *DSF*s are computed based on the absolute acceleration response history recorded at the roof of a building. The *DSF*s are then used as *EDP* indicators by monitoring the change in wavelet-based *DSF*s conditioned at a seismic intensity. The techniques considered in this paper are briefly described in the following subsections.

2.1 Wavelet-based damage sensitive features

Given a scale parameter $a > 0$, and time shift parameter b, the continuous wavelet transform (CWT) can be mathematically described as follows,

$$C(a,b)=\int_{-\infty}^{\infty} f(t)\frac{1}{\sqrt{a}}\psi^{*}\left(\frac{t-b}{a}\right)\mathrm{d}t \tag{1}$$

in which, $f(t)$ is the response history data; $\psi(\mathrm{t})$ is the mother wavelet function (the Morlet wavelet basis function [22] is used as a mother wavelet due to its resemblance to earthquake pulse); and * is the complex conjugate. A set of basis functions, which are termed as daughter wavelets, is established by continuously dilating and translating the mother wavelet function, $\psi(t)$. The CWT coefficients, $C(a, b)$ are then obtained by convoluting the basis functions and response history data, $f(t)$ (e.g., recorded absolute acceleration response history at the building roof). In order to calculate the wavelet-based *DSF*s, the normalization method for wavelet energy at its first mode natural frequency is used as proposed in [21]. Therefore,

$$DSF = 1-\frac{E_{\mathrm{scale}(\hat{a})}}{E_{\mathrm{tot}}} \tag{2}$$

in which, $E_{\mathrm{scale}(\hat{a})}$ is the wavelet energy at scale $\hat{a}$ over time as defined in [23]. This energy can be computed as follows,

$$E_{\mathrm{scale}(\hat{a})} = \sum_{b=1}^{K}\left|C\left(\hat{a}, b\times\Delta t\right)\right|^{2} \tag{3}$$

the total wavelet energy, E_{tot} of the acceleration response history is the sum of the wavelet energies over time at the scales $\hat{a}$, and $2\hat{a}$ that correspond to the first and half of the first natural frequency of the building under consideration, respectively (i.e., $\hat{a}$ is the scale when pseudo-frequency of the daughter wavelet is equivalent to the first natural frequency of undamaged state). The *DSF* values range between 0 (representing no structural damage) and 1 (representing severe structural damage) as suggested in [21].

2.2 Application of wavelet-based *DSF*s to experimental data

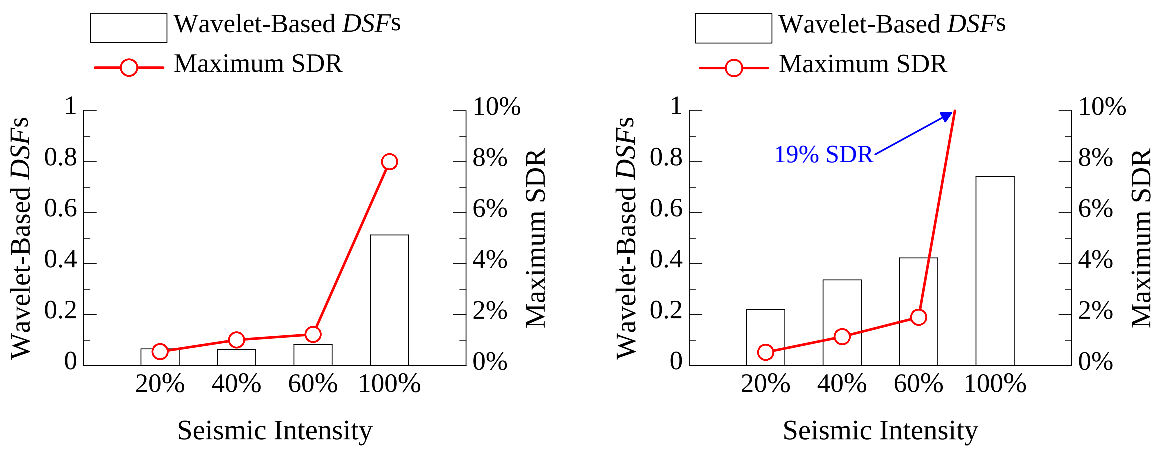
The use of wavelet-based *DSF*s as structural damage indicators is validated with data from three large-scale shake table experiments [24-28]. In these tests, the seismic behavior of the steel frame buildings is well documented from minor damage through the occurrence of structural collapse. Figure 1 shows one of the three case studies utilized for this purpose. It is a full-scale four-story steel frame building with MRFs tested at the E-Defense shake table facility. The steel MRFs were designed in accordance with the Japanese standards [29, 30]. The three components (two horizontal and one vertical) of the JR Takatori ground motion recorded during the 1995 Kobe earthquake was simultaneously applied as an input to the shake table. The test structure experienced three seismic intensities prior to structural collapse including 0.2, 0.4, 0.6 of the unscaled JR Takatori record. Finally, at 1.0 of the original JR Takatori record the building collapsed after about 7 seconds of ground motion shaking. Details of the test results of the test structure can be found in [24, 27, 28].

Figure 2 illustrates the wavelet-based *DSF* values of the four-story building with respect to the seismic intensity in the two principal loading directions (i.e., X- and Y-directions). In the same figure, the additional vertical axis illustrates the maximum story drift ratio (SDR) with respect to the seismic intensity. From Fig. 2(a), the difference of wavelet-based *DSF*s between the 20% and 40% seismic intensities are minor. This is due to the fact that the test structure remained elastic during the 20% JR Takatori record and experienced minor yielding in its first story columns during the 40% and 60% scaled intensities (e.g., maximum SDR in the X-direction was 1.01% at the second story during 40% motion). From Fig. 2, it is interesting to note that the *DSF*s between the

60% and 100% seismic intensities drastically change in both loading directions. During the 100% of the unscaled JR Takatori record, the test structure collapsed with a first story sidesway mechanism (i.e., 19% SDR) before it rested on the safeguard system shown in Fig. 1(a). From Fig. 2, the wavelet-based *DSF* increases while the SDR demands that the test structure experiences increase. Therefore, the wavelet-based *DSF* is strongly correlated with peak SDRs along the height of the four-story building. This is consistent with earlier findings as discussed in [20, 21]. Similar findings hold true for the rest of the shake table experiments that were investigated [25, 26]. It was also found that wavelet-based *DSF*s are well-correlated with peak floor absolute accelerations (PFAs) and residual SDRs along the height of the steel frame buildings that were evaluated.

(a) Test setup (b) Plan view

Fig. 1 – Full-scale collapse test of a four-story steel frame building with MRFs (adopted from [24])



(a) X-direction (b) Y-direction

Fig. 2 – Wavelet-based *DSF*s for four-story steel frame building at E-Defense facility

3. Proposed method for simplified assessment of steel frame buildings with MRFs

In order to propose an approximate method for simplified assessment of steel frame buildings with MRFs, we developed a database of inelastic building seismic responses obtained from NRHA. Based on results the relationship between the story-based *EDP*s in the database and the wavelet-based *DSF*s is established. The approximate method is developed based on stepwise multivariate linear regression analysis. Story-based *EDP*s are predicted within few minutes based on the respective building height, the ground motion intensity and the wavelet-based *DSF* that is computed based on the recorded absolute acceleration response history at the building roof. Therefore, the use of a detailed nonlinear building model is not required.

3.1 Database development

In order to populate the data of the best-suited damage indicators (i.e., wavelet-based *DSF*s), data from archetype building collapse simulations are utilized. The archetypes range from 2 to 20 stories and their steel MRFs are designed for three strong-column/weak-beam (SCWB) ratios of 1.0 (code-based design), 1.5 and 2.0. The design and seismic behavior of these buildings has been discussed in detail in [31, 32]. Figure 3 illustrates a plan view and elevation of a representative four-story steel frame building with perimeter MRFs. The best-suited damage indicators are used for the development of empirical equations for the computation of story-based *EDP*s without the use of detailed nonlinear building models.

Two-Dimensional (2-D) analytical model representations of the east-west (E-W) bare MRFs (see Fig. 3) are developed in the Open System for Earthquake Engineering Simulation (OPENSEES) Platform [33]. The MRF steel beams and columns are modeled with elastic elements and concentrated plasticity flexural hinges at their ends. The phenomenological deterioration model that was developed by Ibarra *et al.* [34] and further refined and calibrated by Lignos and Krawinkler [35], is utilized for this purpose. To represent the hysteretic behavior of a beam-to-column joint panel zone, a *parallelogram* model is used as discussed in [36]. Second order effects (i.e., P-Delta effects) are considered in the analytical model.

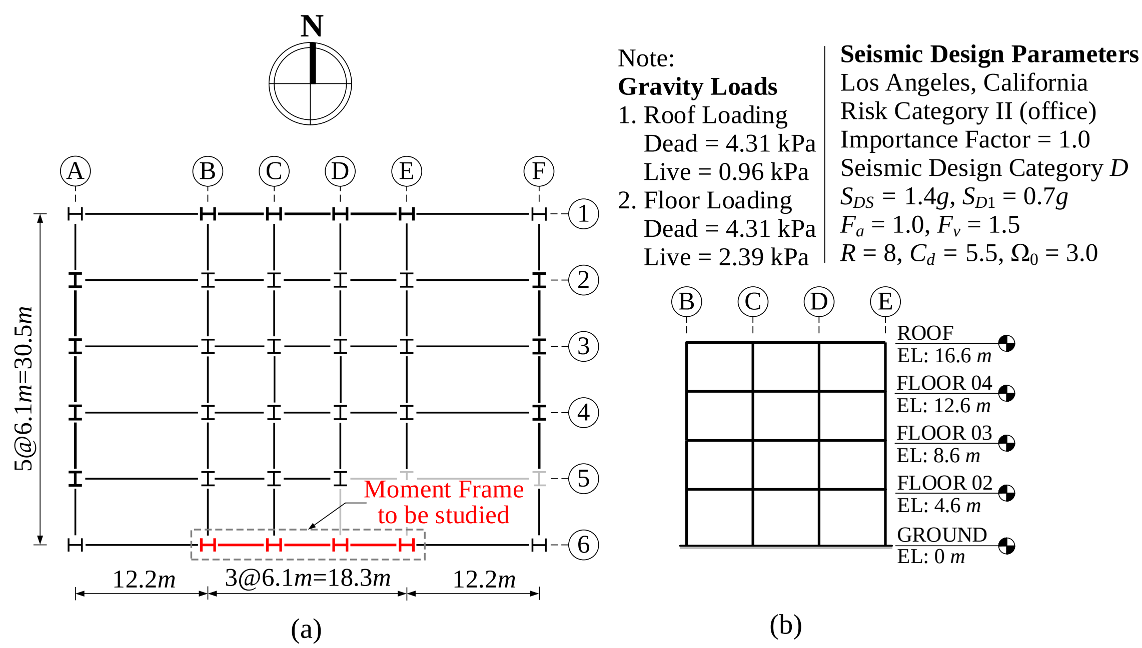


Fig. 3 –Typical archetype steel frame buildings: (a) plan view; and (b) elevation of the four-story steel MRF

Multiple NRHA [i.e., incremental dynamic analysis (IDA) [4] are performed based on a suite of ground motions with large moment-magnitude, $6.5 \leq M_w \leq 7$ and short closest-to-fault-rupture distance, 13 km < R_{rup} < 40 km (LMSR–N set) compiled by Medina and Krawinkler [37]. Different story-based *EDP*s of interest such as peak SDRs, residual SDRs, and PFAs are obtained for each ground motion over a wide range of seismic intensities. The wavelet-based *DSF*s are determined from the absolute acceleration response histories recorded at the roof of each archetype as discussed in Section 2. The computed *DSF*s are then used as predictors together with minimal geometric parameters that can be easily retrieved from the building under consideration.

3.2 Proposed empirical equations for rapid earthquake damage assessment

Multivariate linear regression analysis is employed in order to estimate the story-based *EDP*s along the height of steel frame buildings based on the database of building responses discussed in Section 3.1. Based on the stepwise multivariate regression analysis approach [38], statistically significant predictor variables are included in the empirical equations. Equation (4) represents the general form of the empirical equations for estimating median *EDP*s of interest,

$$\ln\left(EDP_i\right) = \beta_0 + \beta_1 \ln\left(IM\right) + \beta_2 \ln\left(DSF\right) + \beta_3\left(h_x/H\right) + \beta_4\left(h_x/H\right)^2 + \beta_5\left(h_x/H\right)^3 + \beta_6\left(SCWB\right) + \beta_7\left(N\right) + \varepsilon \quad (4)$$

in which, β_i are the regression constants; ε is the random error; EDP_i are the peak SDRs, residual SDRs and PFAs at each level i; the seismic intensity measures of the ground motion *IM*s are the S_{avg} and *PGA* for estimates of peak SDRs and residual SDRs, and PFAs, respectively; the wavelet-based *DSF* is determined from the absolute acceleration response history recorded at the roof of a building; h_x is the height above the base of the building to floor level x; H is the total building height above the ground; *SCWB* is the strong-column/weak-beam ratio determined by the year of building construction; and N is the number of stories of the building under consideration. The range of applicability of Eq. (4) for the estimation of *EDP*s is: $0.02 \leq S_{avg}/g \leq 2.0$; $0.05 \leq PGA/g \leq 8.0$; $0 \leq DSF \leq 0.85$; $0.13 \leq h_x/H \leq 1.0$; $1.0 \leq SCWB \leq 2.0$; and $2 \leq N \leq 20$. It was found that the average spectral acceleration S_{avg} proposed by Eads *et al*. [39, 40] provides better estimates of peak SDRs and residual SDRs compared to other IMs that were examined. Similarly, the peak ground acceleration (PGA) provides better estimates for PFAs compared to other IMs that were considered. Note that the FEMA P-58 simplified approach [2] utilizes the same IM for computing PFAs.

To treat the statistical error and uncertainty in the regression model, *t*- and *F*-statistics are performed at a 5% significance level. The estimated values of the regression coefficients are summarized in Tables 1 and 2, along with the coefficient of determination R^2, the standard deviation $\sigma_{\ln}$ and the coefficient of variance (COV), for buildings with less than 8-stories and buildings with 9 to 20 stories, respectively.

Table 1 – Regression coefficients for story drift ratio, peak absolute floor acceleration, and residual story drift ratio for steel frame buildings with less than 8 stories

***EDP*s**	β_0	β_1	β_2	β_3	β_4	β_5	β_6	β_7	***IM***
Peak SDR	1.52	0.72	0.04	1.41	–1.23	–0.08	0.00	–0.02	S_{avg}
	$R^2 = 0.682$, $\sigma_{\ln} = 0.366$, COV=0.141								
PFA	0.33	0.47	0.16	–0.11	0.00	0.00	0.06	–0.01	*PGA*
	$R^2 = 0.691$, $\sigma_{\ln} = 0.271$, COV=0.204								
Residual SDR	0.33	0.55	0.05	–0.18	0.00	0.00	–0.11	0.03	S_{avg}
	$R^2 = 0.361$, $\sigma_{\ln} = 0.563$, COV=0.901								

Table 2 – Regression coefficients for story drift ratio, peak absolute floor acceleration, and residual story drift ratio for steel frame buildings buildings with 9 to 20 stories

***EDP*s**	β_0	β_1	β_2	β_3	β_4	β_5	β_6	β_7	***IM***
Peak SDR	1.40	0.60	0.04	3.76	–7.50	4.05	–0.08	–0.005	S_{avg}
	$R^2 = 0.563$, $\sigma_{\ln} = 0.392$, COV=0.173								
PFA	0.56	0.51	0.16	–1.03	–0.84	0.00	0.02	–0.005	*PGA*
	$R^2 = 0.716$, $\sigma_{\ln} = 0.270$, COV=0.192								
Residual SDR	0.354	0.416	0.012	0.750	–1.01	0.00	–0.137	0.009	S_{avg}
	$R^2 = 0.219$, $\sigma_{\ln} = 0.582$, COV=0.881								

3.3 Evaluation of proposed method for predicting *EDP*s in steel frame buildings with MRFs

In this section, the efficiency of the proposed empirical equations in predicting story-based *EDP*s in steel frame buildings with MRFs is evaluated with respect to results obtained from rigorous NRHA. An 8-story steel frame building with MRFs designed in downtown Los Angeles (33.996°N, 118.162°W) is used as a case study. The building is subjected to the far-field set of 44 ground motions from FEMA P695 [41]. For comparison purposes the simplified procedure summarized in FEMA P-58 [2] is also considered. The comparison is done for three discrete levels of intensity of interest to the engineering profession: namely, (a) service-level earthquake (SLE, seismic hazard level of 50% probability of exceedance in 50 years); (b) design-basis earthquake (DBE, 10% probability of exceedance in 50 years); and (c) maximum considered earthquake (MCE, 2% probability of exceedance in 50 years) as defined for the design location of interest.

Figure 4 illustrates the predicted peak SDRs along the height of the 8-story steel frame building based on the proposed equations in comparison with the median response based on NRHA for all three levels of seismic intensity of interest. In the same figure, we have superimposed the predicted median peak SDRs based on the FEMA P-58 simplified procedure [2]. On the basis of the FEMA P-58 procedure, we determined the median SDR by using (a) a nonlinear building model; (b) an elastic analysis based on a first-mode lateral force distribution; and (c) an estimate of the building's lateral yield strength by conducting a nonlinear static analysis based on a first mode lateral load pattern. From Fig. 4, it is evident that the proposed predictive equations provide reasonable estimates of peak SDRs along the height of the building regardless of the seismic intensity. Note that the only input information that is required is the building height, the employed SCWB ratio and the computed *DSF* based on the absolute acceleration response history at the roof of the 8-story building. The FEMA P-58 simplified procedure provides slightly better estimates of median SDR responses compared to that of the proposed method for the SLE and DBE seismic intensities [see Figs. 4(a) and 4(b)]. This is mainly due to the fact that this procedure uses a detailed analytical model that appropriately represents the distribution of mass and stiffness along the height of the building. The building's mode shape and lateral yield strength is also utilized. From Fig. 4c, the FEMA P-58 simplified approach significantly overestimates the peak SDRs in the upper stories of the 8-story building at the MCE intensity. This is due to the fact that this approach is not applicable when SDRs exceed 4 times the corresponding yield drift ratio and excessive deterioration in strength and stiffness of structural components occurs. Based on Fig. 4(c), the proposed method predicts fairly well the *EDP*s of interest.

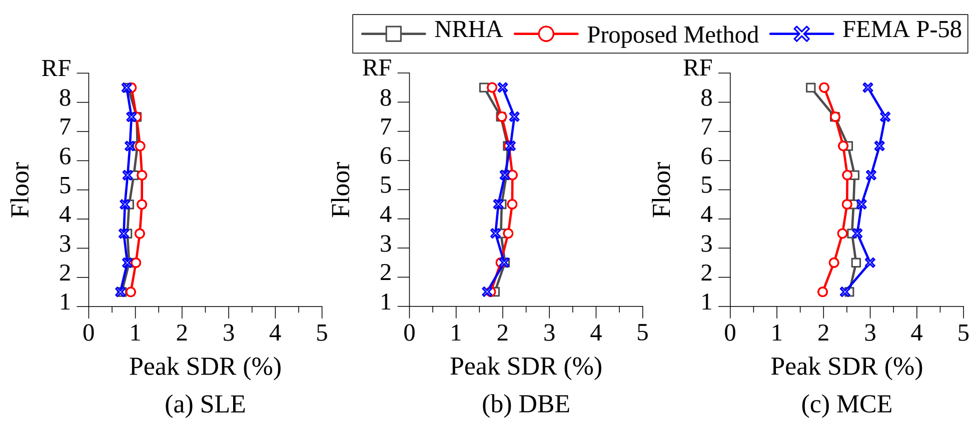


Fig. 4 – Predicted versus simulated peak story drift ratios along the height of the 8-story steel frame building with SCWB > 1.0

The predicted median PFAs along the height of the 8-story building are shown in Fig. 5 for the three levels of seismic intensity of interest based on the proposed method and the FEMA P-58 simplified approach. Superimposed in the same figure is the median response based on results from NRHA. From this figure, it is found that the proposed method provides reasonable PFA estimates along the height of the building for moderate seismic intensities (i.e., SLE, DBE). For seismic intensities with low probability of occurrence [see Fig. 5(c)] even though the proposed method captures the saturation of PFAs due to the nonlinear response of the building, it underestimates PFAs by approximate 15%, on average, compared to NRHA. Similar accuracy is achieved with the FEMA P-58 simplified approach.

Figure 6 compares the predicted residual SDRs along the height of the 8-story building based on the proposed method and the FEMA P-58 simplified approach for the three seismic intensities of interest. In the same figure we have superimposed the median response based on NRHA. From Fig. 6, the proposed method tends to slightly overestimate the residual SDRs around the mid-height of the building. The FEMA P-58 simplified approach tends to underestimate the residual SDRs at the bottom stories of the building [see Figs. 6(a) and (b)]. This is generally not the case for higher seismic intensities associated with MCE [see Fig. 6(c)]. Previous research has identified that residual story drifts are highly variable and very sensitive to the earthquake magnitude, distance to the source range, the adopted component hysteretic behavior as well as the analytical model representations of a building [17, 42-44]. For the aforementioned reasons, a lower/upper bound analysis can be employed based on the $16^{th}/84^{th}$ percentile of the predicted values based on the proposed method. For this reason, the COV values summarized in Tables 1 and 2 can be utilized.

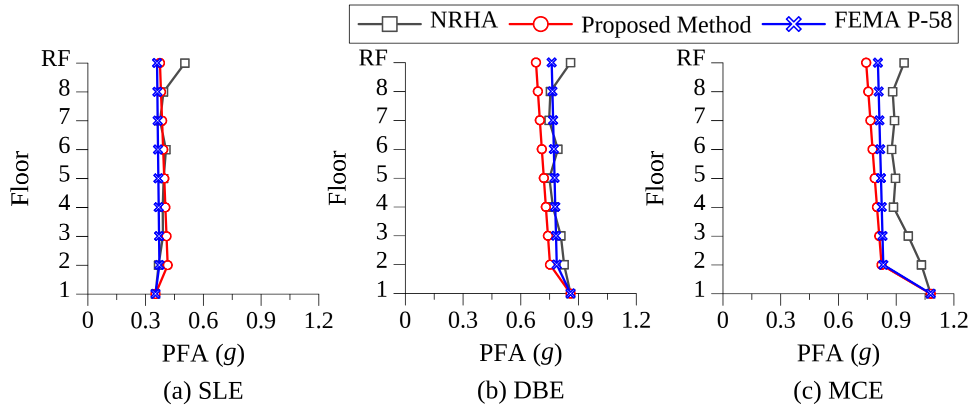


Fig. 5 – Predicted versus simulated peak floor absolute acceleration for 8-story steel frame building with SCWB of 1.0

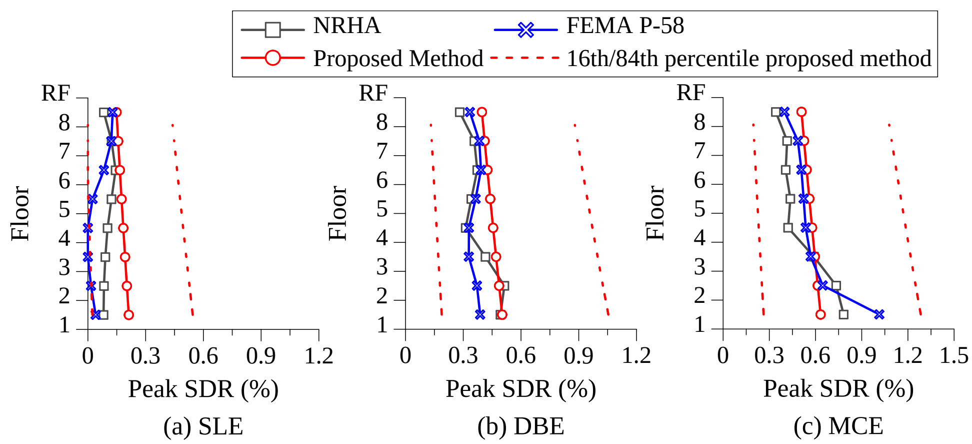


Fig. 6 – Predicted versus simulated residual drift ratio for 8-story steel frame building with SCWB of 1.0

4. Application of simplified assessment methodology in instrumented steel frame buildings

4.1 Case study instrumented building

The proposed method for rapid earthquake assessment of steel frame buildings with MRFs is evaluated with the use of recorded data from an instrumented steel building located in California. This building experienced the 1994 Northridge earthquake. The selected building is the 15-story Government steel frame office building (Station Number: CSMIP 24569) located in Los Angeles, California (34.058°N, 118.250°W). The lateral load resisting system of this building consists of steel MRFs. The building was designed in 1961. Therefore, capacity design principles did not apply in this case. A concrete shear wall exists at the basement level. Fifteen accelerometers were placed at four levels along the height of this building that recorded the building response during the earthquake. The recorded data is retrieved from the Center for Engineering Strong Motion Data (CESMD) operated by the California Department of Conservation's Strong Motion Instrumentation Program (CSMIP) in cooperation with the US Geological Survey (USGS).

4.2 Prediction of engineering demand parameters

To determine the wavelet-based *DSF*s to be used in the proposed equations, the first natural frequency f_1 of the building is identified in its two orthogonal loading directions based on the base motion and output absolute acceleration response histories recorded at the roof of the building during the earthquake. The numerical algorithms for subspace state space system identification (N4SID) technique [45] is used for this purpose. The identified frequencies are summarized in Table 3. In case that the response history at a different floor is only available, the output-only system identification approach developed by Lignos and Miranda [46] can be employed to compute the base and roof motions needed for the proposed method discussed in this paper.

Table 3 – System identification for 15-story instrumented steel frame office building in Los Angeles

Loading direction	**Natural frequency f_1 (Hz)**	**Damping ratio, ξ_1 (%)**
North-South	0.34	2.2
East-West	0.32	3.4

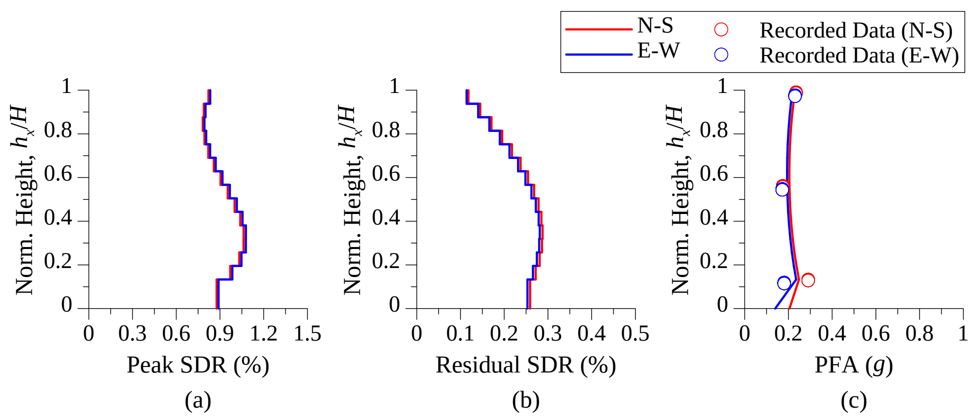


Fig. 7 – Predicted *EDP*s for Los Angeles – 15-story Government office building (CSMIP 24569)

The estimated story-based *EDP*s of interest along the height of the 15-story building are shown in Fig. 7 for both loading directions [North-South (noted as N-S) and East-West (noted as E-W). These *EDP*s are computed within few minutes based on Eq. (4) and Table 2. The wavelet-based *DSF* is determined from the recorded absolute acceleration response history at the roof of the building. Other than that, the total height of the building is only used for such prediction. In Fig. 7(c) the recorded peak floor absolute accelerations at three floor levels have been superimposed for comparison purposes with the proposed method. From the same figure, it is found that the proposed method provides accurate estimates of PFAs for the instrumented building of interest. From Figs. 7(a) and (b), the proposed method predicts that the peak SDR and residual SDR along the height of the same building is 1.1% and 0.3%, respectively. This occurs around the mid-height of the building for both loading directions of interest. Therefore, the steel beams of the building barely yielded and therefore the building is deemed to be lightly damaged. In this case, the FEMA P-58 simplified approach cannot be utilized because the building geometry as well as material properties of the respective structural components is not known.

5. Summary and conclusions

This paper proposed an approximate method for estimating story-based *EDP*s such as peak story drift ratios (SDRs), peak floor absolute accelerations (PFAs) and residual SDRs along the height of steel frame buildings with moment-resisting frames (MRFs) in the aftermath of an earthquake. The proposed method utilizes a wavelet-based damage sensitive feature (*DSF*) based on concepts from structural health monitoring (SHM). The method does not require the use of a detailed analytical model of a building in order to compute its response during an earthquake. Two case studies are used for illustration of the potential use of such method for rapid-earthquake assessment of instrumented steel frame buildings with MRFs. The main findings of the paper are summarized as follows:

- Wavelet-based *DSF*s are able to trace changes in building seismic response due to structural damage without the use of detailed analytical models. This was verified based on data from large-scale shake table experiments that evaluated the dynamic response of steel frame buildings with steel MRFs from the onset of damage through the occurrence of structural collapse.
- The proposed method provides reasonable estimates of peak SDRs and PFAs at various seismic intensities of interest in comparison with results from nonlinear response history analysis (NRHA) of an 8-story steel frame building subjected to a set of 44 ground motion records.
- The efficiency of the approximate method in predicting story-based *EDP*s in steel frame buildings with MRFs is compared with the FEMA P-58 simplified approach [2]. It is found that even though the FEMA P-58 approach utilizes a detailed numerical model representation of the building of interest, it only provides slightly better estimates of peak SDRs and PFAs for moderate seismic events compared to the proposed method presented in this paper. For seismic events with low-probability of occurrence, the proposed method provides *EDP* estimates much closer to reality than the FEMA P-58 approach.
- The proposed method predicts reasonably well the residual SDRs of the case study building for moderate seismic events compared to the FEMA P-58 simplified approach. Large discrepancies are observed between predicted and simulated median residual SDR at seismic intensities with low-probabilities of occurrence. In this case, lower/upper bound analysis may be considered.
- The potential use of the proposed method for rapid seismic assessment of structural damage in steel frame buildings with MRFs is illustrated with the utilization of recorded data of an instrumented steel frame building that experienced the 1994 Northridge. The building seismic response is predicted within few minutes (near real-time) without the use of a detailed numerical model.

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