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# EVALUATION OF PEAK AND RESIDUAL DRIFT DEMANDS IN REGULAR MULTI-STORY STEEL FRAMES SUBJECTED TO SOFT SOIL GROUND MOTIONS

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## ABSTRACT

This paper presents main results of an analytical study aimed at evaluating peak and residual drift demands in six regular multi-story steel frames when subjected to a set of 16 earthquake ground motions recorded on soft soil sites of the San Francisco Bay as well as simulated seismic sequences. The set of records was also scaled to represent different levels of ground motion intensity using an inelastic intensity measure. It was concluded that the frame models subjected to as-recorded soft soil ground motions would reach peak inter-story drift demands between limiting drifts associated to the immediate occupancy (IO) and life-safety (LS) structural performance levels, but residual drift demands are negligible satisfying limiting drifts associated to IO. In addition, it was found that simulated aftershocks could trigger larger peak inter-story drifts in 4- and 8-story rigid frames; however, residual drifts demands are not significantly increased after the mainshock.

## Introduction

It is now widely recognized that local site conditions can have an important effect on the ground motion intensity at a given site. This influence is particularly noticeable in the case of soft soil deposits that give rise to narrow-band ground motions. Evidence of inadequate seismic performance of buildings and other types of structures located in soft soil sites have been documented during various earthquakes (e.g. 1967 Caracas earthquake, 1977 Vrancea, Romania earthquake, 1985 Michoacan earthquake, 1989 Loma Prieta earthquake). For example, soft soil deposits in the San Francisco Bay Area were a key factor in the observed damage during the 1989 Loma Prieta earthquake, including the collapse of a one-mile segment of the Cypress Street Viaduct. In particular, post-earthquake field reconnaissance have shown that some damaged structures may need to be demolished due to excessive permanent lateral deformations (i.e., residual displacements), at the end of the earthquake ground shaking, even if they did not experience severe damage or partial collapse. Thus, the evaluation of residual displacement demands plays a very important role in determining the technical and economical feasibility of repairing and retrofitting structures that have been damaged due to earthquake excitation. In addition, an adequate estimation of residual displacement demands has been shown to be critically important in evaluating the structural residual capacity and in assessing possible collapse during strong aftershocks (e.g. Luco et al. 2004). Thus, an adequate estimation of

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residual displacement demands that existing structures may experience after earthquake ground shaking should be of primary importance in modern performance-based assessment procedures.

Motivated by earthquake field reconnaissance observations, researchers have performed analytical investigations recently aimed at gaining further understanding on the parameters that influence the amplitude and height-wise distribution of residual drift demands in existing multi-story buildings (e.g. Pampanin et al. 2002, Ruiz-García and Miranda 2006). Previous studies have reported that the residual drift demand amplitude and distribution over the height depends on the component hysteretic behavior, building frame mechanism, structural overstrength as well as the ground motion intensity. Particularly, Ruiz-García and Miranda (2006) noted that the evaluation of residual drift demands in regular moment-resisting frame models involves large levels of uncertainty (i.e. record-to-record variability) in its estimation and, moreover, this uncertainty is larger than that associated to the estimation of maximum (transient) drift demands.

However, previous studies have evaluated residual drift demands at the end of ordinary medium-to-broad band earthquake ground motions (i.e. having medium-to-high frequency content) recorded on accelerographic stations placed either on rock or on firm soil sites. Thus there is a lack of information about the amplitude and height-wise distribution of residual drift demands in multi-story frames subjected to narrow-band ground motions such as those recorded on soft soil conditions (e.g. in the San Francisco Bay Area or the old bed-lake area of Mexico city). Furthermore, recognizing that a structure is actually subjected to a sequence of mainshock-aftershocks, it is of particular interest to study if strong aftershocks could increase permanent displacements at the end of the mainshock seismic excitation.

The primary objective of this paper is to present relevant observations of an analytical study aimed at gaining further understanding on the parameters that influence the amplitude and height-wise distribution of peak and residual drift demands in multi-story frames subjected to earthquake ground motions recorded in accelerographic stations placed on soft soil sites of the San Francisco Bay area. In particular, the influence of fundamental period of vibration and number of stories, as well as the effect of aftershocks on central tendency of peak and residual drift demands is discussed in this paper.

## Frame models, earthquake ground motions and intensity measure

### Multi-story frame models

For the purpose of evaluating drift demands, two families of regular three-bay frame models having three different number of stories (N = 4, 8, and 12), which are representative of exterior steel moment resisting frames found in typical existing steel office buildings, were considered in this investigation. The frame models were originally designed by Santa-Ana and Miranda (2000) for computing strength reduction factors of multi-degree-of-freedom (MDOF) systems. The first family comprises stiff frame models, with periods of vibration between 0.71 and 1.53 s, while the second family includes flexible frame models, with periods of vibration between 1.23 and 2.61 s. This distinction allows studying seismic response of frame models with the same number of stories, but different fundamental periods of vibration. Both families of generic frames were designed according to lateral load distribution prescribed in the 1994

Uniform Building Code. The frames were modeled as two-dimensional centerline models, as illustrated in Figure 1, using the computer program RUAUMOKO (Carr 2008). Rayleigh damping equal to 5% of critical was assigned to the first and fourth mode for all frame models. It should be mentioned that global $P-\Delta$ effect was considered, but element $P-$ delta effect was neglected. Beams and columns were modeled with frame elements which concentrate the inelastic response in plastic hinges at both ends of the frame elements. In the column plastic hinge, non-degrading elastoplastic moment-curvature hysteretic relationship as well as axial load-flexural bending interaction was considered. An elastoplastic moment-curvature hysteretic relationship including strength degradation was considered in the beam elements for simulating fracture according to Filiatrault et al. (2001). Flexural moment capacity for all elements was determined from actual yield strength capacity equal to 45 ksi for steel grade A-36.

Table 1. Fundamental period of vibration, $T_1$, roof yield displacement, $\delta_{yroof}$, yield strength coefficient, $C_y$, and normalized modal participation factor, $\Gamma_1\phi_1$, obtained for each generic frame considered in this investigation.

| | $T_1$ [sec] | | $\delta_{yroof}$ [cm] | | $C_y$ | | $\Gamma_1\phi_1$ | |
|---|---|---|---|---|---|---|---|---|
| **N** | **Rigid** | **Flexible** | **Rigid** | **Flexible** | **Rigid** | **Flexible** | **Rigid** | **Flexible** |
| 4 | 0.71 | 1.23 | 15.0 | 16.0 | 0.88 | 0.32 | 1.21 | 1.22 |
| 8 | 1.18 | 1.92 | 33.0 | 31.0 | 0.58 | 0.25 | 1.31 | 1.31 |
| 12 | 1.53 | 2.61 | 35.0 | 47.0 | 0.39 | 0.18 | 1.27 | 1.31 |

## Ground motion ensemble and simulation of seismic sequences

A core part of the results reported in this paper were obtained from non-linear time-history dynamic response of the family of generic frame building models when subjected to a set of 16 mainshock acceleration time histories recorded in accelerographic stations located on bay mud sites in the San Francisco Bay Area during the 1989 Loma Prieta earthquake (Ms= 7.1). The San Francisco Bay is located in a basin about 15 km wide bounded by the active San Andreas and Hayward fault zones. This region is characterized by a wide variety of geologic deposits from rocks sites in the hill area to estuarine mud and clay deposits in the flatlands along the margins of the bay. The bay mud area is comprised of unconsolidated, water-saturated, dark plastic clay and silty clay with well-sorted silt and sand dunes in some areas. It may contain more than 50% of water content and low shear-wave velocities in the range of 67 to 116 m/s. The list of 16 earthquake ground motions can be found in Ruiz-Garcia and Miranda (2004).

In order to evaluate the influence of aftershocks in the response of structures, a set of as-recorded seismic sequences mainshock-aftershock representative of an earthquake hazard environment should be available. For example, Ruiz-García et al. (2008) have identified an ensemble of 26 seismic sequences representatives of the seismic hazard in the subduction zone of the Mexican Pacific coast. In the absence of as-recorded seismic sequences, simulation of seismic sequences using a set of mainshocks has been employed (e.g. Li and Ellingwod 2007). However, it should be noted that repeating the mainshock as aftershock represent an unbelievable scenario, since not only the ground motion intensity of the mainshock-aftershock pair is the same, but also their frequency content and ground motion duration. Recognizing this fact, seismic sequences for soft soil conditions were simulated in this study by combining one mainshock of the ground motion ensemble with the remaining mainshocks, which is named to as a 'randomization' approach by Li and Ellingwod (2007). As an example of this simulation procedure, a simulated seismic sequence using the acceleration time-histories gathered in the Emeryville (comp. 260) and Larkspur Ferry Terminal (comp. 270) recording stations is shown in Fig. 1. Therefore, 240 seismic sequences were simulated from the ground motion dataset considered in this study. It should be mentioned that in some simulated sequences, the peak ground acceleration of the aftershock is greater than the peak ground acceleration of the mainshock, which has been observed in as-recorded seismic sequences (Ruiz-García et al. 2008).

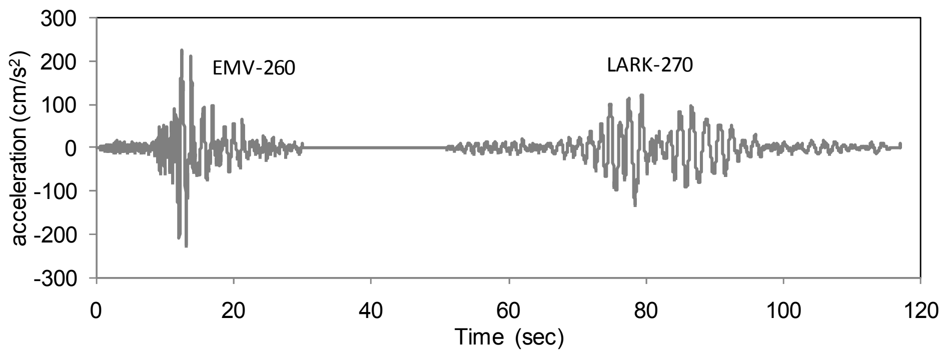


Figure 1. Simulated seismic sequence mainshock-aftershock for the San Francisco Bay Area.

**Inelastic intensity measure**

Of particular interest to this investigation was the estimation of peak and residual drift demands of the study-case frame models under a set of ground motions at different levels of intensity. This task was accomplished by using the so-called Incremental Dynamic Analysis (Vamvatsikos and Cornell 2002). An important component in this procedure is the selection of an appropriate parameter to characterize the intensity of the ground motion, which is also known as intensity measure, $IM$. Previous studies (e,g. Ruiz-Garcia and Miranda 2006) have noted that an inelastic intensity measure which consists on scaling ground motions to reach the same maximum inelastic displacement of an equivalent SDOF system having the same initial lateral stiffness (i.e., fundamental period of vibration) and yield displacement of the building of interest, $\Delta_y$, leads to smaller record-to-record variability than other proposed $IM'$s. Therefore, in this study it was decided to use a relative inelastic $IM$ to scale each ground motion. This relative inelastic $IM$ is defined as $\eta = \Delta_i(T_1)/\Delta_y$, where $\Delta_y$ is the yield displacement of the equivalent SDOF system. The yield displacement of the equivalent SDOF system can be related to the global (i.e., roof) yield displacement of the structure, $\delta_{yroof}$, by normalizing it by the product of the modal participation factor and mode amplitude at the roof corresponding to the building's first-mode of vibration, $\Gamma_1 \varphi_1$ (i.e. $\Delta_y = \delta_{yroof}/\Gamma_1 \varphi_1$). Thus, the relative inelastic $IM$ used in this investigation is expressed as follows:

$$IM = \frac{\Delta_i(T_1)}{\delta_{y,roof}/\Gamma_1\varphi_1} \qquad (1)$$

Roof yield displacements for each generic building model, were determined using nonlinear static (pushover) analyses using a parabolic lateral load pattern and were also performed using RUAUMOKO (Carr 2008). All acceleration time histories were scaled to have the same maximum inelastic displacement demand of an equivalent elastoplastic SDOF system with the same fundamental period of vibration of the structure and corresponding to five target relative inelastic $IM$'s ($IM$= 0.5, 0.75, 1.0, 2.0, 3.0). Each building model is expected to behave in the elastic range for relative intensities smaller than about 0.9 and expected to experience nonlinear behavior for relative intensities larger than about 1.1. It should be noted that for the range of relative intensities considered in this study most buildings would not experience extremely large inelastic deformations, so no dynamic instabilities are expected.

## Results of statistical study

### Response under as-recorded mainshock ground motions

Figs. 2 and 3 show the height-wise distribution of peak (transient), $IDR$, and residual (permanent), $RIDR$, interstory drift ratio demand for the 4-story stiff and flexible frame models, respectively under as-recorded (dashed line) and scaled (continuous line) ground motions. Although both frames develop a 'soft' story, it can be seen that the flexible frame exhibits larger peak drift demands than its rigid counterpart under both as-recorded and scaled ground motions. For instance, under unscaled ground motions, peak inter-story drift demand in the first story of the flexible frame is greater than the rigid frame about 50%. It can be seen that for both frames peak inter-story drift demands are between the limiting drifts prescribed in FEMA 356 (2000) recommendations for the immediate occupancy, IO, (0.7%) and life-safety, LS, (2.5%) performance levels, which mean that some minor yielding or local buckling would be expected. In addition, although residual drift demands are larger for the flexible frame than its rigid counterpart about twice, in both frames the level of residual drift demands is very small that satisfy permanent limiting drifts for IO performance level (i.e. negligible permanent drift). Under scaled ground motions that induce nonlinear behavior (e.g. *IM* equal to 2), peak and permanent drift demands of the 4-story flexible frame are above the limiting peak and permanent drift (i.e. 1.0%) limits of the LS performance level, while the 4-story rigid frame would be still between the IO and LS performance level.

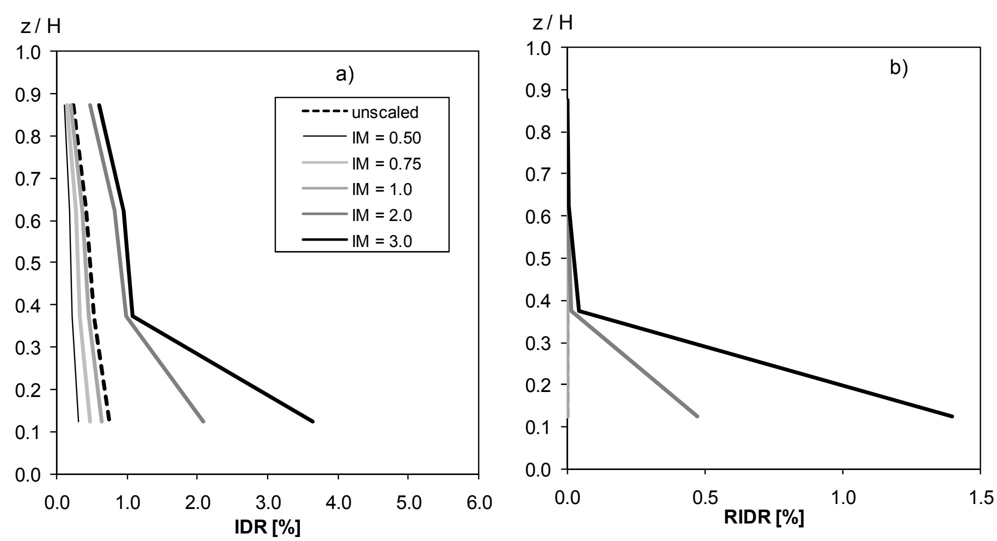


Figure 2. Evolution of height-wise median drift demand in the 4-story stiff frame model: a) peak (transient), b) permanent (residual).

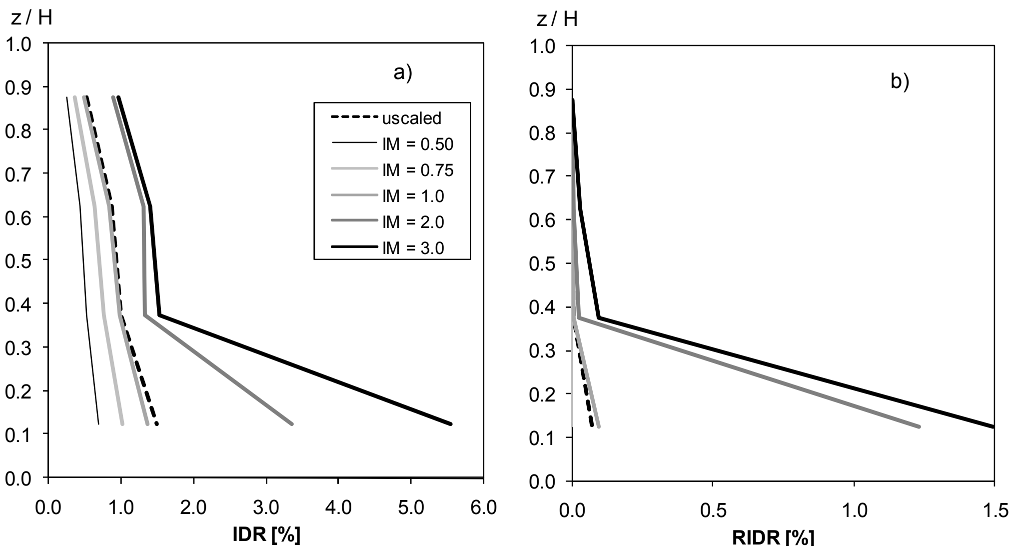


Figure 3. Evolution of height-wise median drift demand in the 4-story flexible frame model: a) peak (transient), b) permanent (residual).

Next, Figs. 4 and 5 show the height-wise distribution of peak (transient) and residual (permanent) interstory drift ratio demand for the 8-story stiff and flexible frame models, respectively. Under as-recorded ground motions, it can be seen that the flexible frame suffer larger peak drift demands than its rigid counterpart, particularly in the upper stories. This could be explained since three of the ground motions that trigger the largest peak interstory drifts ratios (LARK-360, LARK-270, TREASI-90) have predominant period of the ground motion close to the fundamental period of the flexible frame, which could amplify the drift demand. For both frames, the level of residual interstory drift demands is again very low. Unlike the 4-story frames, the 'soft' story mechanism appears under scaled ground motions

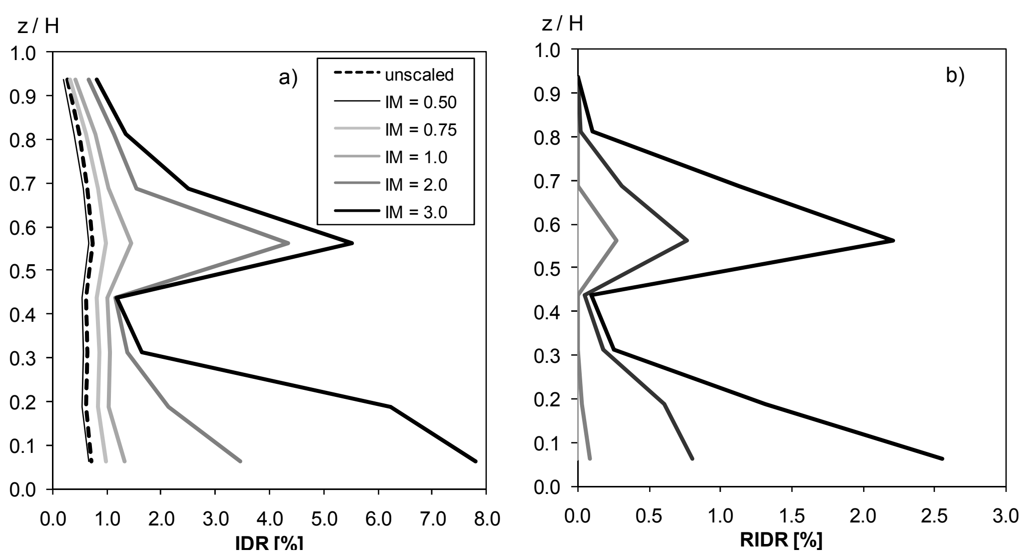


Figure 4. Evolution of height-wise drift demand in the 8-story stiff frame model: a) peak (transient), b) permanent (residual).

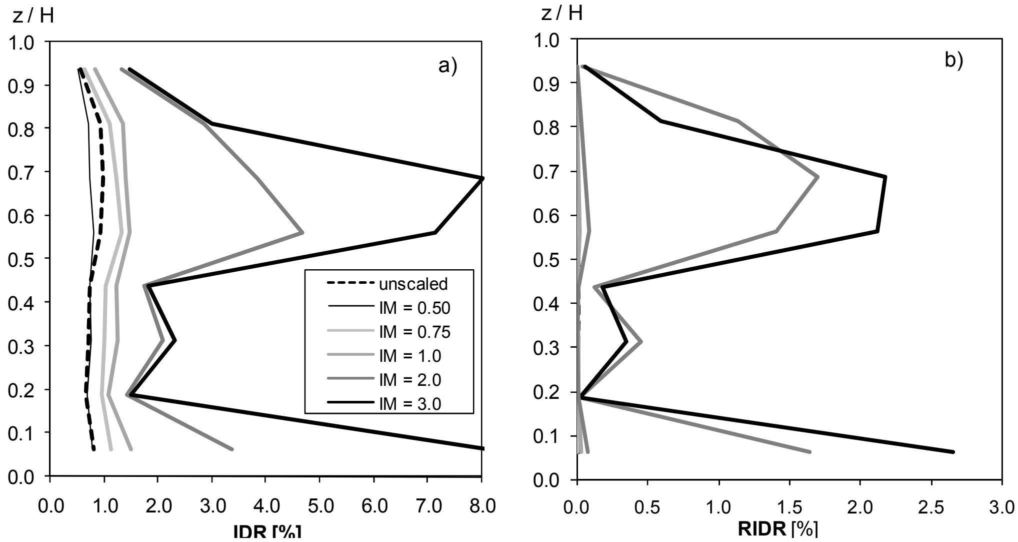


Figure 5. Evolution of height-wise drift demand in the 8-story flexible frame model: a) peak (transient), b) permanent (residual).

Finally, the evolution of both peak and residual drift demands is shown in Figs. 6 and 7 for the 12-story stiff and flexible frame models, respectively. Comparing Figs. 6a and 7a, it can be seen that while the flexible frame experiences larger $IDR$'s in the upper levels than its rigid counterpart, the rigid frame exhibits larger $IDR$'s in the ground story under the as-recorded ground motions. This observation could be attributed to the influence of higher modes in the flexible frame, although periods of vibration associated to the first and second mode are not close to the predominant period of the ground motions. Again, residual drift demands at the end of the earthquake excitation are negligible, which can be explained since only two records induced inelastic response to the frame.

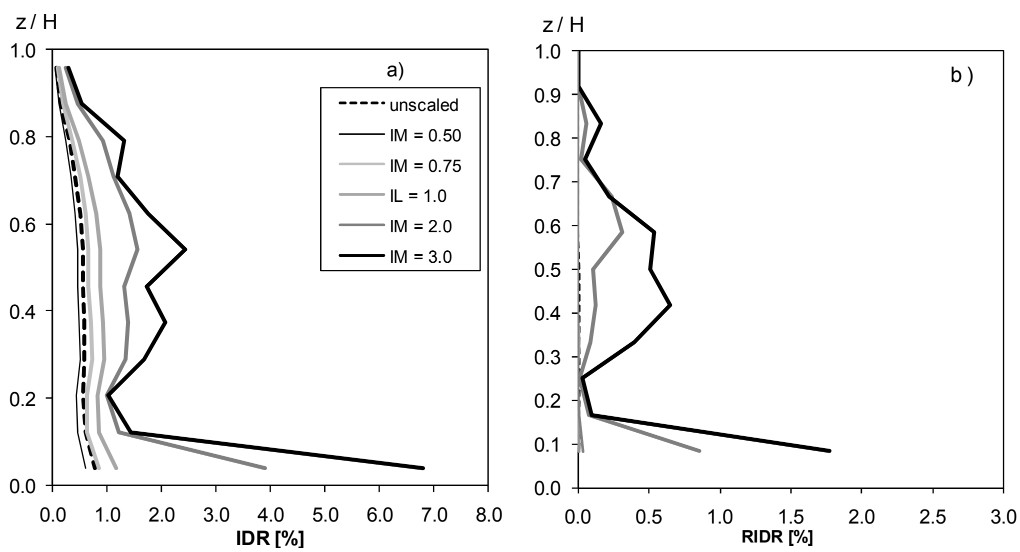


Figure 6. Evolution of height-wise interstory drift demand in the 12-story stiff frame model: a) peak (transient), b) permanent (residual).

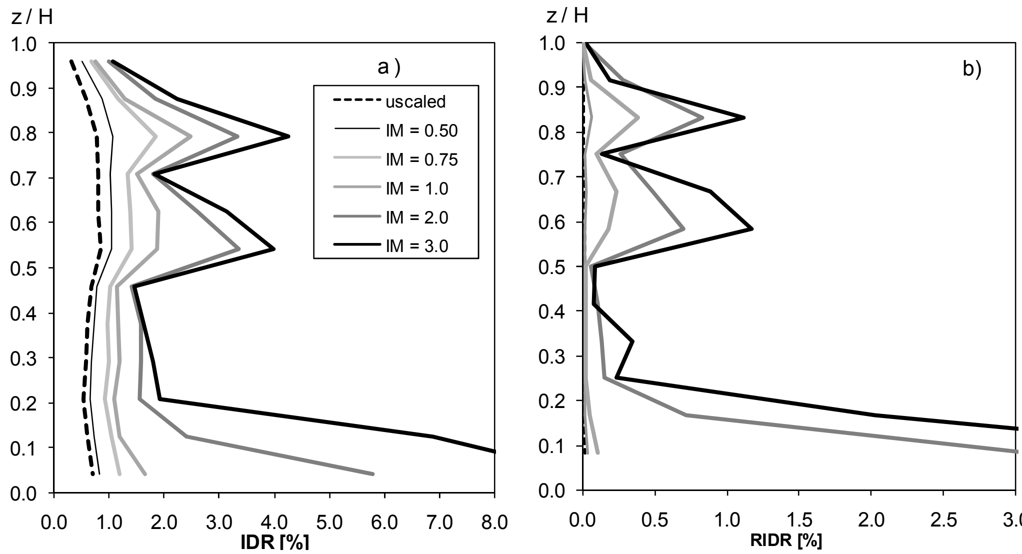


Figure 7. Evolution of height-wise interstory drift demand in the 12-story flexible frame model: a) peak (transient), b) permanent (residual).

**Response under mainshock-aftershock sequences**

Due to space limitations, only the influence of seismic sequences in the response of the flexible 4- and 8-story frame models is discussed. From Fig. 8, it can be seen that, in general, both median peak and residual interstory drift demands experienced in the 4-story frame model would increase as a consequence of the aftershocks, which is more evident in the bottom stories since the frame develop a 'soft' story mechanism. In particular, peak interstory drift demands would increase about 16% in the ground story, while permanent interstory drift demands at the end of the mainshock would increase about 22% as a consequence of the aftershocks in the same story.

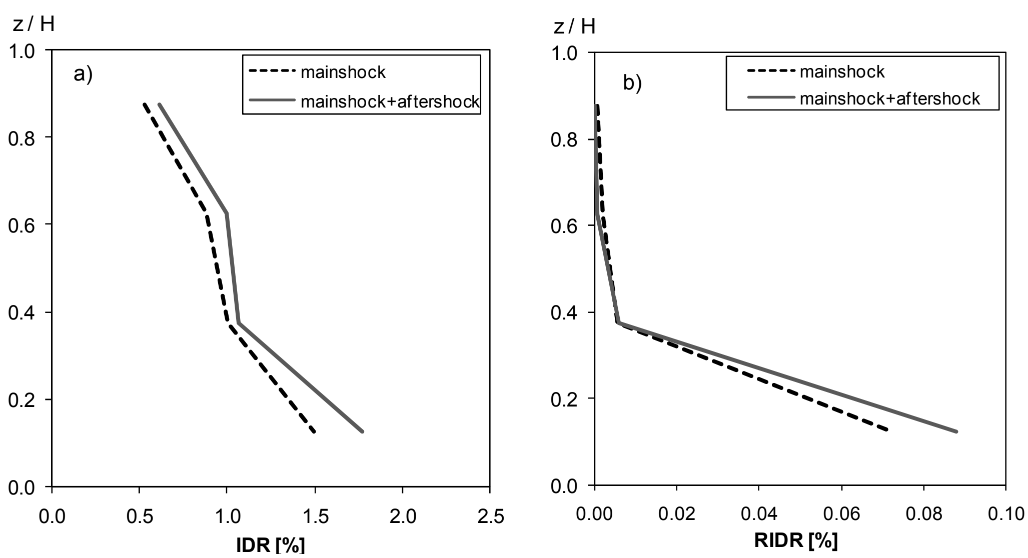


Figure 8. Evolution of height-wise interstory drift demand in the 4-story flexible frame model: a) peak (transient), b) permanent (residual).

On the other hand, Fig. 9 shows the height-wise distribution of median peak and residual interstory drift demands due to the set of as-recorded mainshocks and the set of simulated seismic sequences. It can be seen that the aftershocks could increase peak interstory drift demands (e.g. 32% in the bottom story),  which implies driving the frame above the limiting peak interstory drift levels associated to IO performance level, but it seems that the aftershocks might tend to re-center the frame. Some explanations to this observation are: a) structural elements exhibiting degrading hysteretic features that tend to re-center the hysteretic loops and, as a consequence, to constrain permanent drifts (Ruiz-Garcia et al. 2008), b) unlike the 'short'-flexible frame, the increasing effect of higher modes in the response of the 'tall'-flexible frame could be beneficial to constrain permanent drifts, and c) unlike one-bay tall-flexible frames, redundancy in multi-bay frames could have a beneficial effect to limit permanent drift demands. Therefore, further investigation on the effects of aftershocks on existing structures located on soft soil conditions is highly desirable.

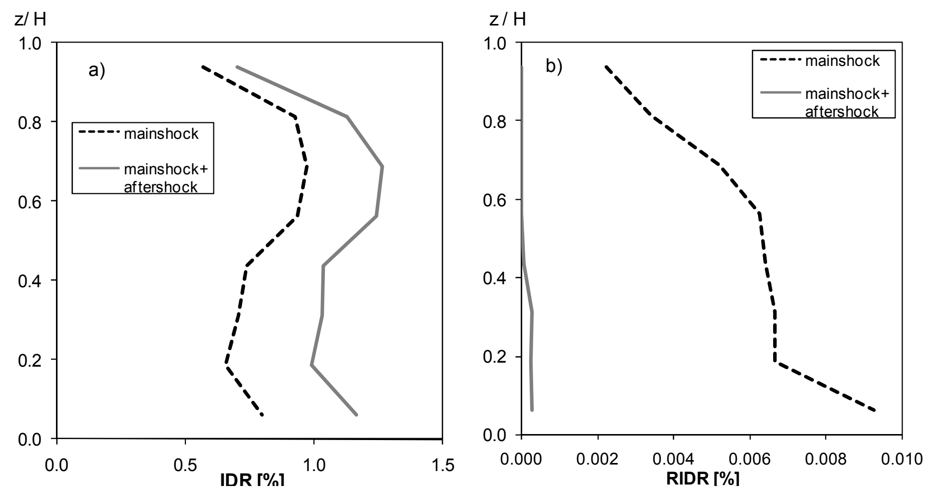


Figure 9. Evolution of height-wise interstory drift demand in the 8-story flexible frame model: a) peak (transient), b) permanent (residual).

## Conclusions

The purpose of this study was to evaluate peak (transient) and residual (permanent) drift demands in multi-story steel frames subjected to 16 mainshock earthquake ground motions recorded on soft soil sites of the San Francisco Bay as well as simulated mainshock-aftershock seismic sequences. From this investigation, it was found that, in general, the steel frame models considered in this investigation would reach peak inter-story drift demands between 0.7% and 2.5% (i.e. between limiting drifts associated to the IO and LS structural performance levels prescribed in FEMA 356) under the set of as-recorded soft soil ground motions considered in this study, but residual drift demands were found negligible.

Using simulated mainshock-aftershock seismic sequences, it was found that the aftershocks could trigger larger peak and residual interstory drift demands in the 4-story flexible frame model, which develop a 'soft' ground story mechanism, than those demands from the mainshock. Although the simulated seismic sequences could trigger larger peak interstory drift demands in the 8-story flexible frame than that from the mainshock, it could be re-centered at the end of the aftershocks. This observation could be explained due to the re-centering capability of the hysteresis loops, the influence of higher mode effects, and the effect of redundancy.


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