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Effect of column base flexibility on residual drift demands of low-rise steel moment-resisting frames

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ABSTRACT

This paper addresses the effect of column base flexibility on residual drift demand assessment of modern low-rise steel moment-resisting frames. Seismic performance was measured by both transient and residual (i.e. permanent) drift demands. For this purpose, two exterior steel moment-resisting frames of 2 and 4 stories designed with recent American Standards were modeled and analyzed under a set of earthquake ground motions of increasing intensity. Each frame was modeled with three types of column base flexibility (fixed, pinned and expected by tuning values of the rotational stiffness of linear springs that represent the base flexibility). Furthermore, refined modeling of the frame elements which included the panel zone flexibility and cyclic deterioration of the frame elements was considered in this study. Results obtained in this investigation show that the height-wise distribution of both drift demands is significantly influenced by the assumed column base flexibility. For ideal pinned-based condition, drift demands concentrate in the bottom stories as opposite to the fixed- and expected-condition. Furthermore, in general, ideal pinned-based condition lead to larger transient inter-story drift demands, but smaller residual drift demands, than those of the fixed- and expected-base condition. It is demonstrated that for performance-based assessment, expected column base flexibility should be used instead of ideal support conditions.

1. INTRODUCTION

Nowadays, recently proposed performance-based seismic design and assessment procedures for new and existing buildings emphasize on the estimation of peak lateral drift demands. However, earthquake field reconnaissance have evidenced that residual lateral displacement demands after earthquake excitation (e.g. residual roof drift ratio or maximum residual inter-story drift ratio) also play an important role in defining the seismic performance of a structure and it can have important consequences (e.g. the level of residual drift demand would drive the decision of demolishing a building even if

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it did not experience excessive structural damage). For example, Iwata (2006) noted that the cost of repair leaned steel buildings due to the 1995 Hyogo-Ken-Nambu (Kobe) earthquake linearly increased as the maximum and roof residual drift increased. For this reason, they suggested that steel buildings should be limited to maximum and roof residual drift limits of about 1.4% and 0.9%, respectively, to satisfy a repairability limit state that meet both technical and economical constraints. It should be noted that steel buildings are prone to experience significant residual drift demands due to their inherent energy-dissipating features. Concerned about this issue, a recent study showed that ductile steel moment resisting frames (SMRF) designed under seismic loading following the ASCE 7-05 standard (ASCE 2005) would experience excessive residual inter-story drifts when subjected to earthquake ground motions representative of firm and rock sites scaled to reach the Maximum Credible Excitation level (Erochko 2011). More recently, Bojórquez (2013) highlighted that modern SMRF designed for soft soil conditions (i.e. subjected to low-frequency narrow-band earthquake ground motions) would experience undesirable residual drift demands (e.g. larger than 0.5%) if they experience peak (transient) interstory drift demands in excess of 3.0%.

A common feature in studies related to the estimation of residual drift demands and, in general, seismic performance of SMRFs is that the numerical models assumed that the columns are fixed at their base. However, several studies have noted that the column base flexibility might have an important effect on the seismic response of SMRFs (e.g. Maan 2002, Aviram 2010, Zareian 2013). For instance, a very recent study compared the effect of column base flexibility on the collapse potential of four steel moment-resisting frames having different heights and designed with a modern seismic code (Zareian 2013). Unlike previous studies, their investigation considered the expected column base flexibility in exposed base plate columns and embedded base columns. They noted that the 4-, 8-, and 12-story frames modeled with expected column base flexibility would experience larger seismic response (e.g. larger collapse potential) than that when the ideal fixed-base column fixity is assumed during nonlinear time-history analyses. On the contrary, the 2-story frame would lead to conservative seismic response if it is analyzed with pinned-base columns, as it is customarily in American practice, instead of using expected column base flexibility.

An interesting issue that was not examined in previous studies, including that of Zareian (2013), is the effect of column base flexibility on residual drift demands. Therefore, the primary objective of research reported in this paper is to assess the effect of column base flexibility on residual drift demands of low-rise modern steel moment-resisting frames. For this purpose, the same 2- and 4-story frames described in Zareain (2013) were analyzed under two sets of earthquake ground motions.

2. STEEL FRAMES CONSIDERED IN THIS STUDY

2.1 Building description

Two regular three-bay frame models having different number of stories (N=2 and 4), which are representative of exterior steel moment-resisting frames found in modern low-to-medium height steel office buildings were considered in this investigation. The frames were originally designed as part of the ATC76 project (ATC76 2011). The design base shear strength was determined from the ASCE 7-05 provisions (ASCE 2005) for structures located in site class D in the Los Angeles area. The buildings were designed assuming a Seismic Design Category D_{max} and they belong to Performance Groups PG-1RSA (2-story frame) and PG-2RSA (4-story frame). Figure 1 illustrates the geometry and steel sections of the analyzed frames. Detailed description of the design process and sizing of structural elements can be found in (ATC-76 2011).

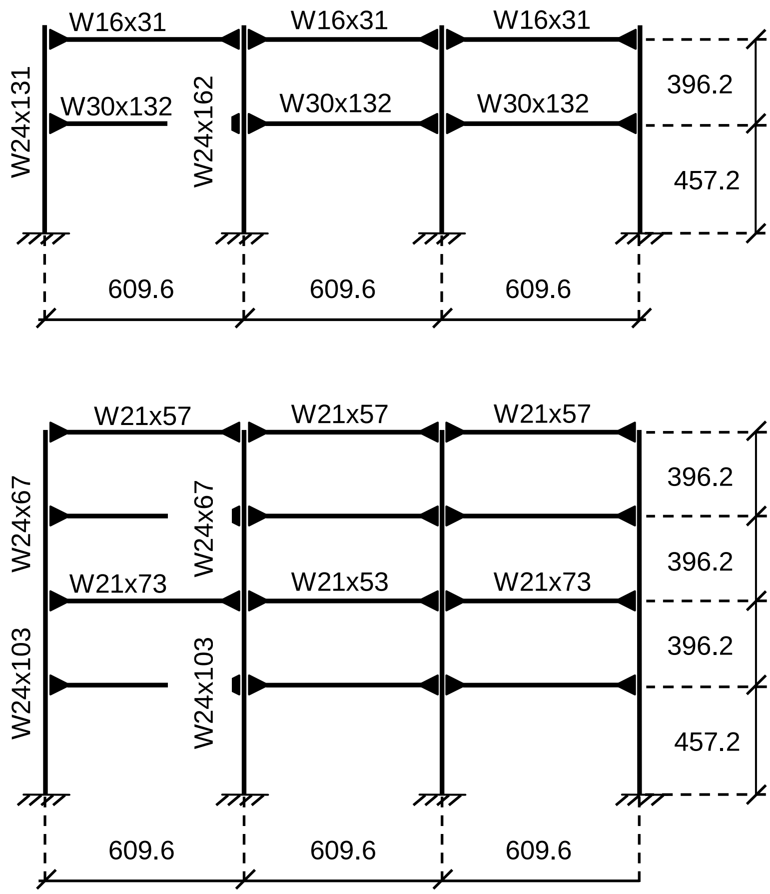

Fig.1 Elevation of the 2-story and 4-story perimeter SMRF considered in this investigation (units in cm)

2.2 Modeling

The buildings were analyzed using the computational platform OpenSees (McKenna 2000). Only half of the building was modeled due to symmetry in the building's plan. The exterior frame was modeled as two-dimensional (2D) centerline model with an additional fictitious column. The fictitious column carries the vertical (gravity) loading from the rest of building (i.e. vertical loading carried by the interior gravity columns) and is attached to the exterior frame model through rigid frame elements to experience the same lateral deformation at each floor. However, the fictitious column does not provide the additional lateral stiffness from the interior gravity columns. Beams and columns were modeled as two-dimensional, prismatic beam elements composed of an elastic beam element with semi-rigid rotational springs at the ends that concentrates their inelastic behavior (i.e. moment-rotation hysteretic behavior) according to what has been discussed in Zareian (2009). The hysteretic behavior in the rotational springs accounts for structural degradation (i.e. cyclic strength and stiffness degradation) using the modified Ibarra-Krawinkler (IK) model implemented in OpenSees platform (McKenna 2000). The parameters of the backbone curve in the IK model for beams were obtained following the expressions proposed by Lignos (2011) corresponding to beams with RBS connections. In addition, panel zone flexibility was taken into account in each frame.

Dynamic time-history analyses were carried out using Newmark constant average acceleration method with time step equal to 0.001s to enhance convergence. Rayleigh damping equal to 3% of critical was assigned to the first and second modes. During the analysis, local P-delta effects were included (i.e. large displacement analysis).

2.3 Modeling of column base flexibility

Column base flexibility was incorporated in the numerical models through rotational springs at the ground base. The rotational springs have linear moment-rotation relationships with three different levels of stiffness to mimic ideal pinned- and fixed-base conditions as well as the expected rotational stiffness. For the latter cases, very small and very large stiffness yield the ideal base conditions, while for the former base condition the values reported in Zareian (2013) were employed in this study. Zareian (2013) computed the expected rotational stiffness of the 2-story frame assuming exposed column base plate connections, while it was assumed that the 4-story frame would have embedded base plate connections. Table 1 reports the fundamental period of vibration (T_1) and the second-mode period of vibration (T_2) corresponding to each of the analyzed frames.

Table 1. Periods of vibration corresponding to each study-case frames

Model	Base flexibility	T_1 [s]	T_2 [s]
2-Story	Pinned	0.97	0.23
	Expected	0.87	0.23
	Fixed	0.59	0.19
4-story	Pinned	2.34	0.63
	Expected	1.76	0.56
	Fixed	1.61	0.52

Before conducting nonlinear response history analysis, the capacity curve for each frame and each case of column-base flexibility was obtained from nonlinear static analysis as illustrated in Fig. 2. From the figure, it can be seen that the influence of column base flexibility in the yield strength capacity is important, manly in the 2-story frame. However, the influence in the roof drift at significant yielding is minor.

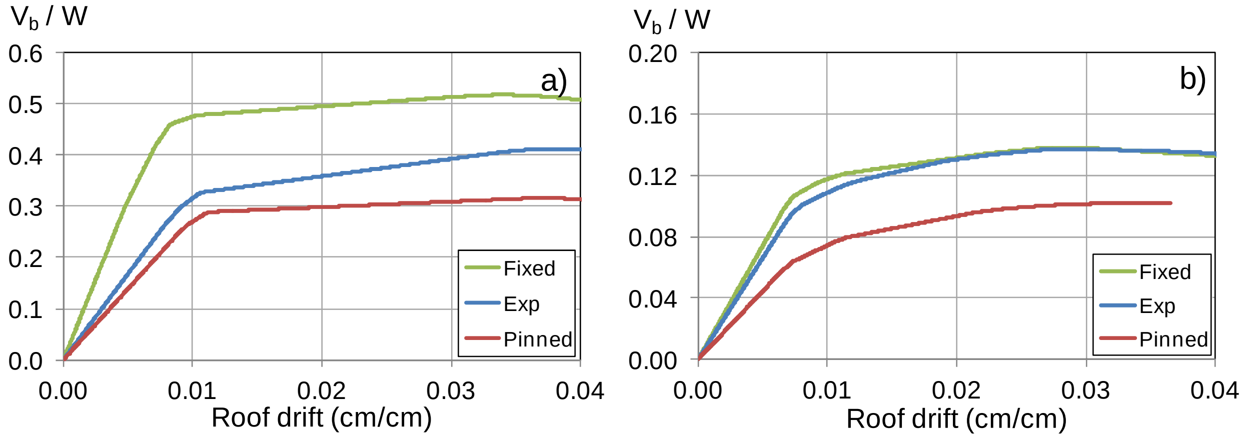

Fig. 2 Capacity curves of the study-case buildings: a) 2-story, b) 4-story

3. EARTHQUAKE GROUND MOTIONS CONSIDERED IN THIS STUDY

A core part of the results reported in this paper were obtained from non-linear time-history analysis of the frame building models when subjected to two sets of earthquake ground motions. The first set contains 22 acceleration time histories recorded in far-field stations. This set of is a subset of far-field earthquake ground motions assembled for the ATC-63 project to assess the collapse safety of modern reinforced concrete buildings (FEMA 2009). The second set is subset of the LMSR-N ground motion set originally assembled by Medina and Krawinkler (2003). All earthquake ground motions included in the LMSR-N set were recorded on stiff soil or soft rock corresponding to soil type D according to FEMA 368 document. This set comprises motions recorded in earthquake events with moment magnitude ranging from 6.5 to 7.0 and with source-to-site distances ranging from 13 km to 40 km, which it can be considered representative of a typical moderate and large magnitude-small distance seismic environment in California. Details of the earthquake ground motion selection (e.g., filter cut-off, etc.) and relevant seismological parameters (e.g., peak ground acceleration, duration of the ground motion, etc.) can be found in Medina and Krawinkler (2003). However, it should be noted that none of the records considered in this study exhibit pulse-type near-fault characteristics, which might trigger larger residual drift demands than those computed from these sets of ground motions, and further research on this issue is recommended.

4. EFFECT OF COLUMN BASE FLEXIBILITY ON SEISMIC PERFORMANCE

4.1 Results under individual ground motion

In American practice, it is commonly assumed that low-story steel buildings are designed under the assumption that ground columns are pinned. However, as discussed in Kanvinde (2012) and Zareian (2013), this assumption neglects the expected column base flexibility and, therefore, might misunderstand its seismic behavior. At a first stage, the effect of the column base flexibility was assessed in the analyzed frames under selected earthquake ground motions. Fig. 3 shows a comparison of the height-wise distribution of peak inter-story drift ratio (IDR) and residual inter-story drift ratio (RIDR) of the 2-story frame when subjected to the G03090 record. From Fig. 3, it can be seen that the ideal pinned-based condition lead to the largest IDR and RIDR demand, which they concentrate in the bottom story. The distribution of IDR along the height of IDR suggests that the type of column base fixity (pinned or fixed) might change the frame mechanism since the largest IDR for the fixed-base column case occurs in the upper story, which is consistent with previous studies. It is interesting to note that the drift profile corresponding to the expected column flexibility is similar to that of the pinned-based condition, but with shorter IDR (e.g. about 75% of that computed from the pinned-based case) and RIDR (e.g. about 35% of that computed from the pinned-based case) in the bottom story.

A similar comparison corresponding to the 4-story frame model is shown in Fig. 4. Again, the pinned-base assumption leads to the largest IDR in the bottom story. However, the largest IDR took place above the bottom story when the expected-based and the fixed-based column based flexibility are assumed in the frame model. Examining the distribution along the height of IDR also suggests that column base flexibility has influence on the frame mechanism (i.e. pinned-based columns leads to a soft-first-story mechanism, while fixed-base and expected-based columns induce plastic hinging above the first story), which is consistent with previous observation from Zareian (2013). However, unlike the 2-story frame, it is interesting to note that pinned-based columns lead to the shortest RIDR, while the fixed-based column fixity assumption triggers the largest RIDR as shown in Fig. 4b. Residual drift demand in the first story for the expected-base column fixity are 2.6 times larger than that of the pinned-based condition, and about 50% shorter than that of the fixed-base assumption.

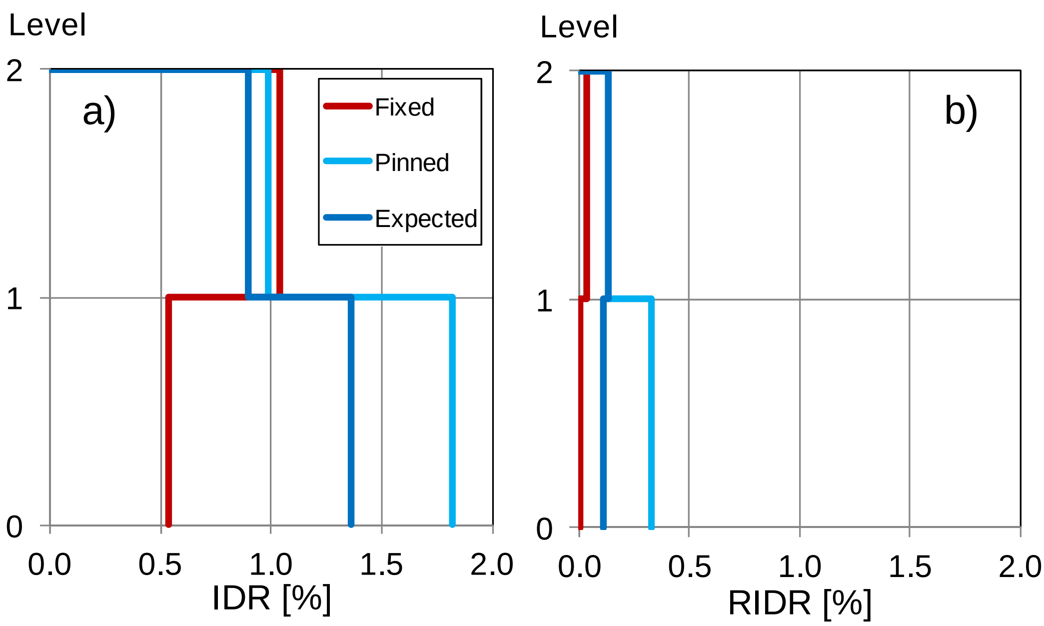

Fig. 3 Comparison of drift demands for the 2-story frame model with three different column base flexibilities when subjected to G03090 record: a) IDR, and b) RIDR

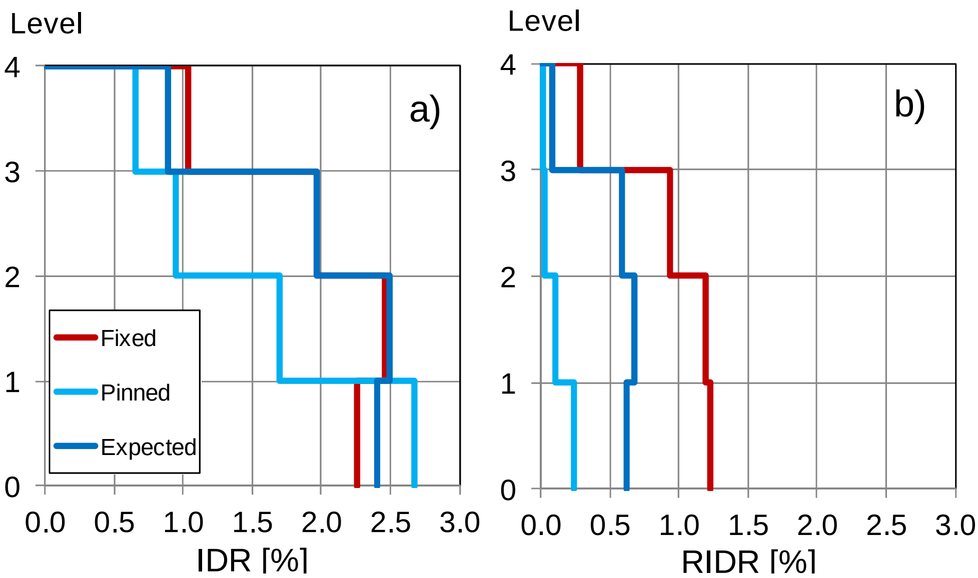

Fig. 4 Comparison of drift demands for the 4-story frame model with three different column base flexibilities when subjected to G03090 record: a) IDR, and b) RIDR

4.2 Results under FF-22 and LMSRN-10 sets

Figs. 5 and 6 show the height-wise distribution of median drift demands (both IDR and RIDR) computed for the 2-story frame under the FF-22 and LMSRN-10 ground motion sets, respectively. In spite of the amplitude, it can be seen that the distribution along height of IDR is very similar for each case under both ground motion sets. Interestingly, the expected column base flexibility lead to the largest residual drift demands under the FF-22 set. On the contrary, the fixed-base assumption lead to the largest residual drift demands under the LMSRN-10 set. These observations might also suggest that the pinned-base columns tend to constrain residual drift demands.

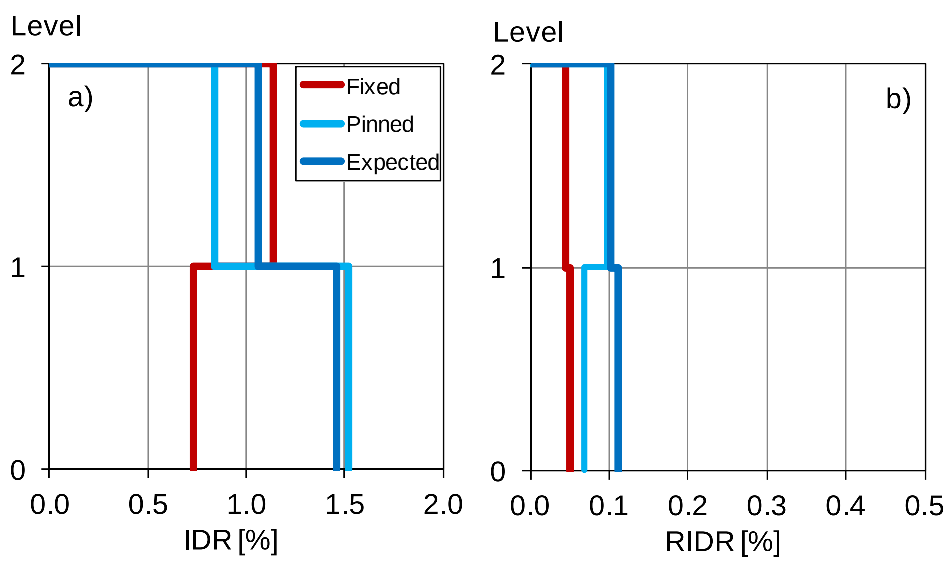

Fig. 5 Median drift demands of the 2-story frame model with three different column base flexibilities computed from the FF-22 set: a) IDR, b) RIDR

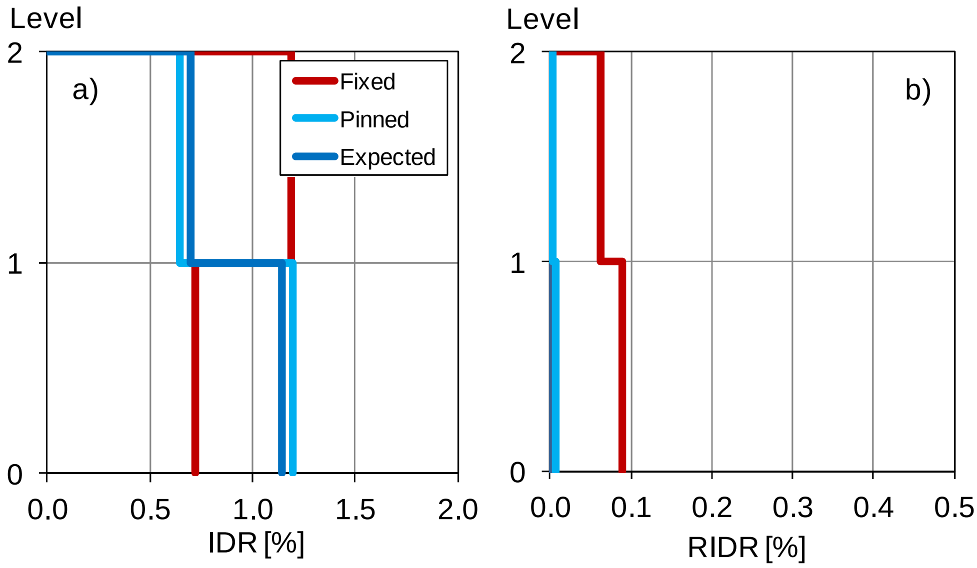

Fig. 6 Median drift demands of the 2-story frame model with three different column base flexibilities computed from LMSR-10 set: a) IDR, b) RIDR

A similar comparison for the 4-story frame under the FF-22 ground motion set is illustrated in Fig. 7. From the figure, it can be seen that the fixed-base column case, which is customarily assumed for design and analysis, trigger smaller peak interstory drift demands in the first story than when a more realistic assumption of the column base flexibility is included in the model. However, the amplitude and height-wise distribution of median residual interstory drift is very similar for the fixed- and expected-cases.

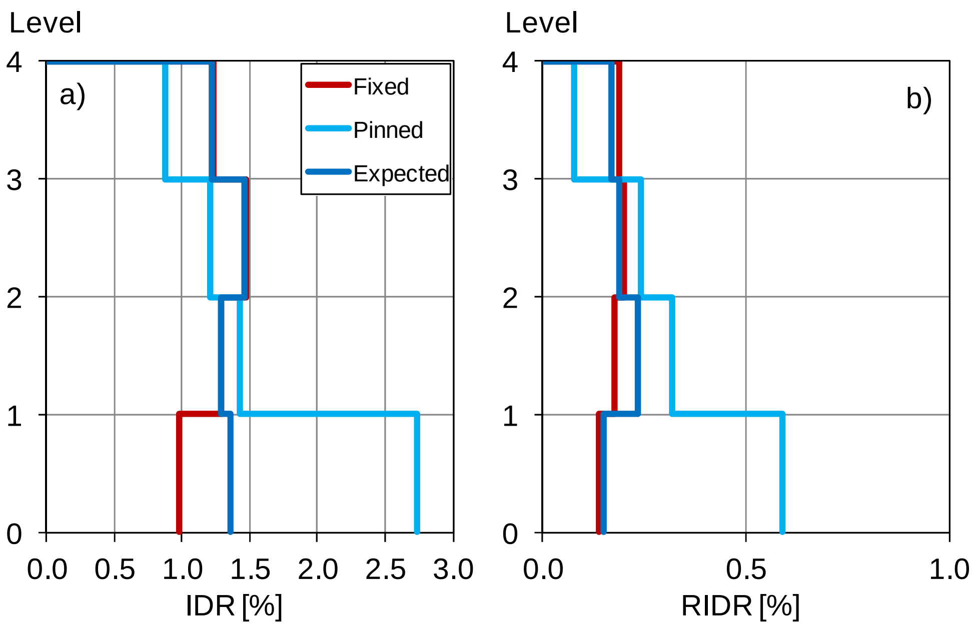

Fig. 7 Median drift demands of the 4-story frame model with three different column base flexibilities computed from the FF-22 set: a) IDR, b) RIDR

4.3 Seismic performance under IDA

Incremental dynamic analysis, IDA (Vamvatsikos 2002), has become a very widely known technique for assessing the seismic performance of structures. Therefore, the effect of column base flexibility of the study-case frames under different levels of ground motion intensity of records included in set LMSR-10 was evaluated in this study using IDA. The spectral displacement at the fundamental period of vibration, S_d (T_1), of each building was chosen as intensity measure. Seismic performance under only five levels of ground motion intensity was examined since collapse safety was not the focus of this study. Figs. 8, 9 and 10 show the evolution of IDR and RIDR for the 2-story frame corresponding to the pinned-base and fixed-base column cases, as well as the expected-base column flexibility case, respectively. From the figures, it can be observed that when ground columns are assumed as pinned, larger IDR's concentrate in the first story as the intensity of the ground motion increases than those corresponding to the fixed case and expected column flexibility. An interesting observation from IDA analyses is that the pinned-base condition would trigger residual interstory drift demands larger than those estimated for the fixed- and expected-cases when the ground motion intensity becomes large, which was not detected under unscaled (i.e. original) records. From these results, it is clear that the assumption of the column base flexibility has an important effect in modern performance-based seismic assessment. Therefore, it is highly recommended to include more realistic assumptions of the column base flexibility for this task.

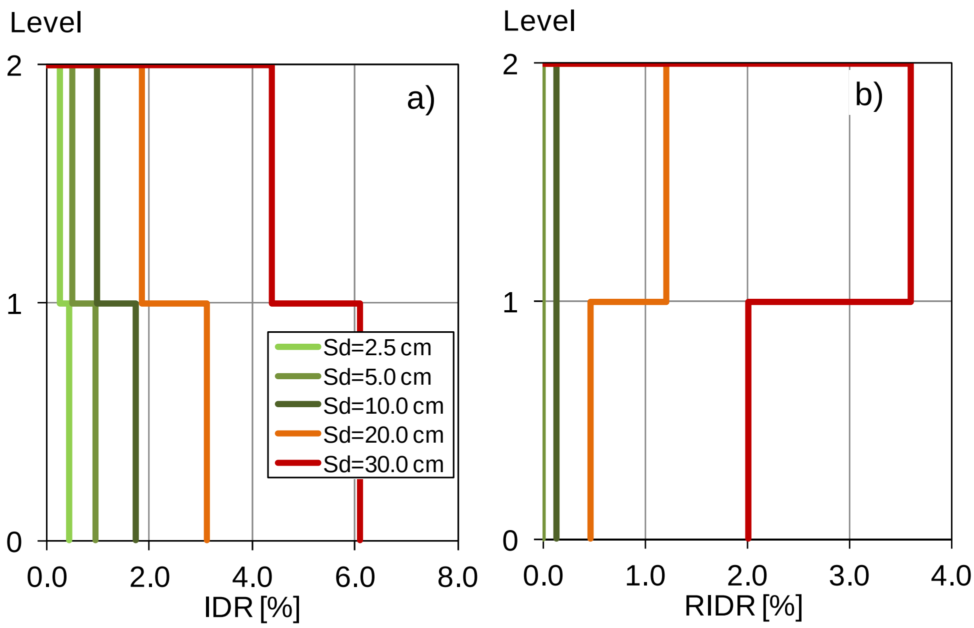

Fig. 8 Evolution of median drift demands of the 2-story frame model with pinned-base ground columns: a) IDR, b) RIDR

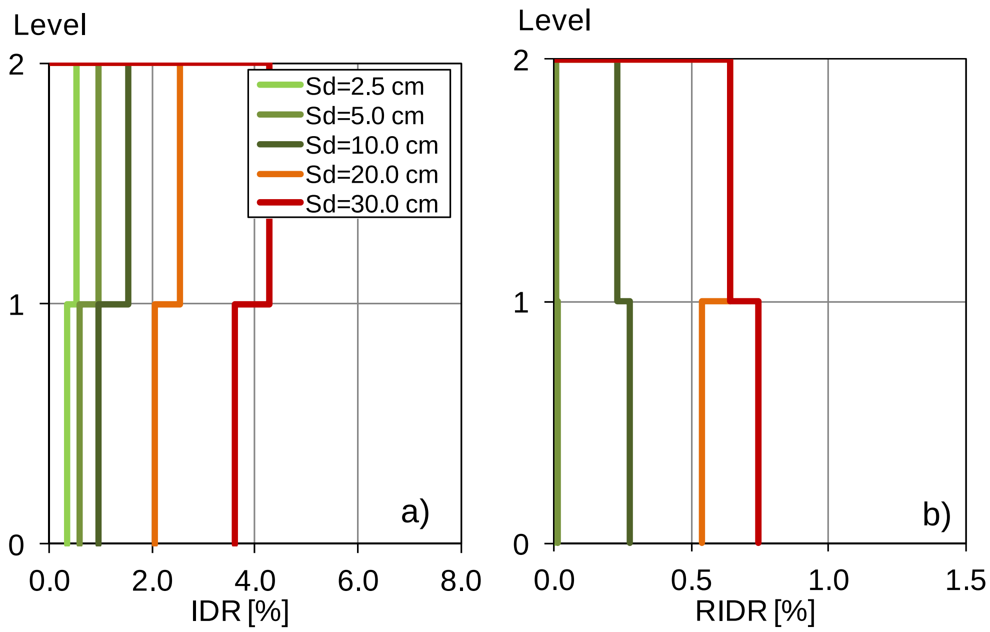

Fig. 9 Evolution of median drift demands of the 2-story frame model with fixed-base ground columns: a) IDR, b) RIDR

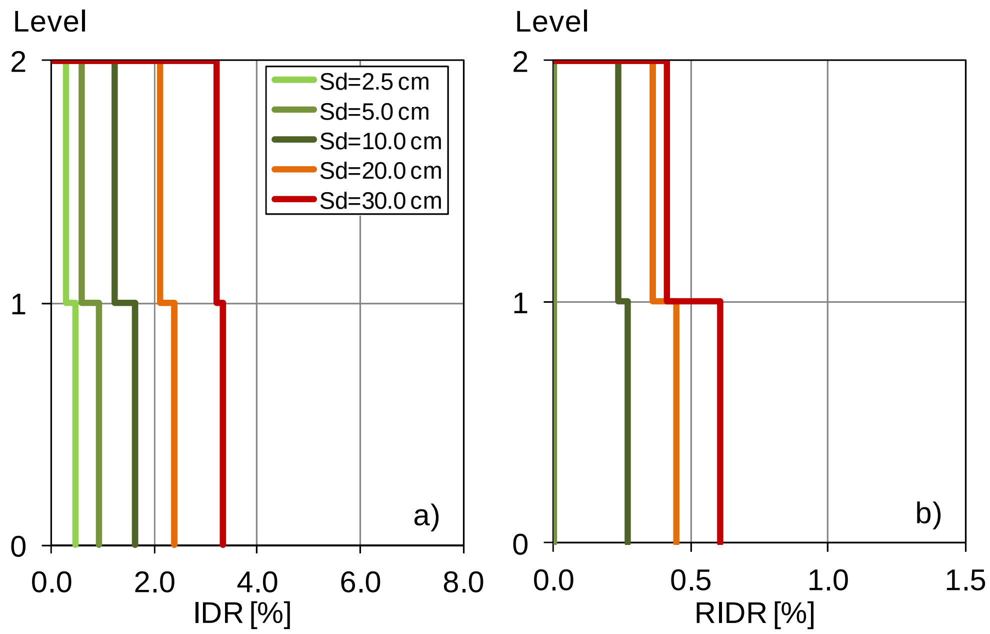

Fig. 10 Evolution of median drift demands of the 2-story frame model with expected column base flexibility: a) IDR, b) RIDR

5. PREDICTION OF RESIDUAL DRIFT DEMANDS

Very recently, the FEMA P-58 (2012) report introduced a criterion for predicting residual drift demands, Δ_r, in its methodology for the seismic performance assessment of buildings. The prediction of is based on the peak transient story drift, Δ, and the story drift at first significant yielding, Δ_y, as follows:

$$\Delta_r = \begin{Bmatrix} 0 \\ 0.3\,(\Delta - \Delta_y \\ (\Delta - 3\,\Delta_y) \end{Bmatrix} \qquad \begin{matrix} \Delta \le \Delta_y \\ \Delta_y < \Delta \le 4\,\Delta_y \\ 4\,\Delta_y < \Delta \end{matrix} \tag{1}$$

Therefore, it is interesting to evaluate the accuracy of FEMA P-58 (2012) for predicting residual drift demands recorded in the study-case steel frames. For instance, Fig. 10 shows a comparison of computed and predicted first-story residual drift demands for the 2-story frame, while a similar comparison for the 4-story frame model is shown in Fig. 11.

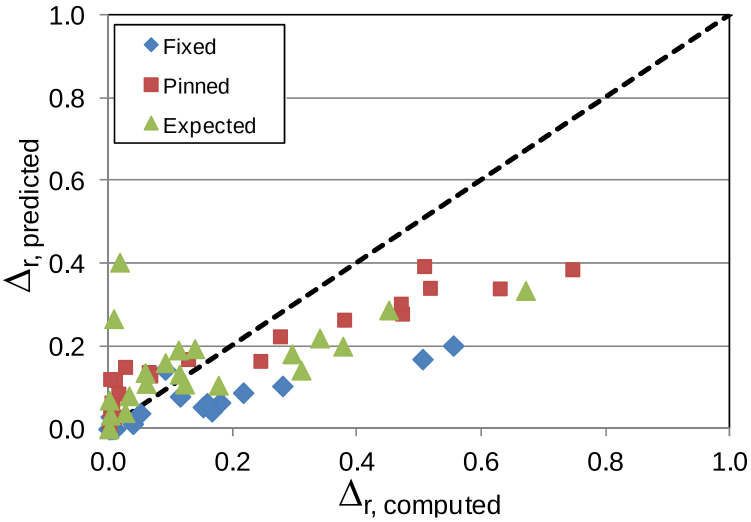

Fig. 11Comparison of predicted and computed residual drift demands for the 2-story frame model

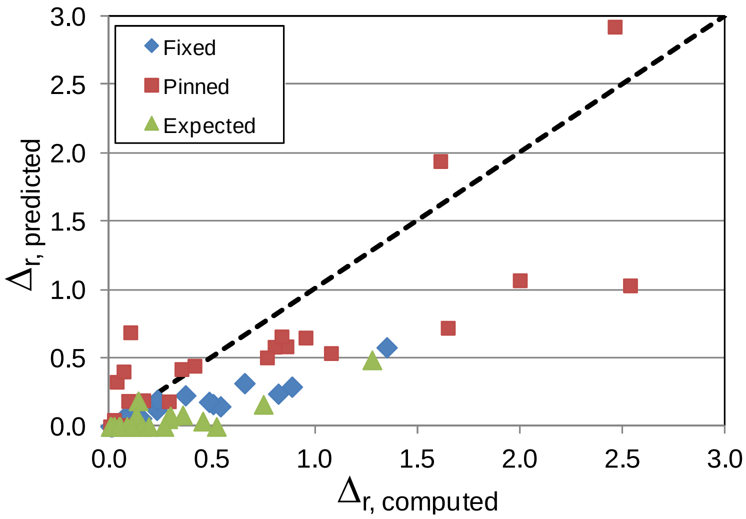

Fig. 12 Comparison of predicted and computed residual drift demands for the 4-story frame model

In general, regardless of the column base flexibility, the prediction of FEMA P-58 tends to underestimate residual drift demands (i.e. points lie below the dashed black line), which is particularly true for the 4-story frame with expected column base flexibility.

6. CONCLUSIONS

The objective of the research reported in this paper was to study the effect of column base flexibility on the residual drift demands of modern low-rise steel moment-resisting frames designed with American standards. Particularly, the amplitude and height-wise distribution of residual drift was examined in this study. The following conclusions are drawn from this ongoing investigation: As noted in previous studies, it was confirmed that the assumption of the column base flexibility has an important impact in the development of the frame mechanism under earthquake excitation. In general, for the 2-story or 4-story frame models, the ideal pinned-base column assumption leads to a ground soft-story mechanism due to drift concentration, while drift demands are distributed along the height when the expected column base flexibility is assumed.

Apparently, the pinned-base column case tends to constrain residual drift demands in the 2-story frame under unscaled earthquake ground motions. However, IDA showed that residual drift demands in the 2-story frames could grow significantly, in the top story if pinned condition is assumed and in the bottom story for the expected condition, as the intensity of the ground motion increases.

A comparison between the computed and predicted residual drift demands from the recently published FEMA P-58 (2012) showed that the methodology tends to underestimate residual drifts, regardless of the type of column base flexibility.

This study showed that the assumption of the column base flexibility is very important for seismic performance-based assessment. In general, it is highly recommended to include more realistic assumptions of the base flexibility while performing nonlinear dynamic analyses.

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