Soil-Structure Interaction effects on the seismic behavior of steel moment resisting frames: Preliminary assessment

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Abstract: The purpose of this paper is to assess the influence of soil-structure interaction and site effects on the seismic behavior of steel moment-resisting frames, with a view to describing the underlying "physics" behind the damage mechanisms triggered. We examine an idealized 2-story, 1-bay steel frame under various earthquake recordings and intensity levels, as well as different subsoil and foundation conditions. The story drift ratio distribution and the moment-rotation response, at the beam and column ends, are indicatively presented and discussed, setting the benchmark for future research. In the cases examined, soil-structure interaction leads to increased drift demands in the first story, while the flexural demands at the fixed end column reduce due to the induced soil flexibility, accompanied by higher inelastic demands at the first-floor steel beams.

Keywords: Earthquake engineering, Soil-structure interaction, Steel moment resisting frames

1. Introduction

To date, the seismic behavior of steel frame buildings along with their vulnerability assessment is thoroughly investigated, involving large sets of nonlinear dynamic analyses (Gupta and Krawinkler 1999, Flores et al. 2014, Elkady and Lignos 2015, El Jisr et al. 2022). However, these studies have neglected the influence of the underlying soil conditions, and particularly soil-structure interaction (SSI). In all cases, ideally fixed-base conditions have been considered for the respective nonlinear building models. It is well established that SSI may be either beneficial or detrimental to the seismic response of structures (Mylonakis and Gazetas 2000), however this influence is yet unquantified, as well as the nonlinear soil behavior effects that come along. This has also been recognized in NIST (2012). Various studies have demonstrated that SSI may significantly alter the structural response in terms of earthquake vulnerability (Rajeev and Tesfamariam 2012, Pitilakis et al. 2013, Behnamfar and Banizadeh 2016, Petridis and Pitilakis 2020; 2021), addressing reinforced concrete buildings. Regarding steel moment resisting frames (MRFs) that form the primary focus of this paper, Mashhadi et. al. (2021) adopted the beam-on-nonlinear-Winkler foundation (BNWF) model to address SSI effects along with different seismic parameters, while Akhoondi and Behnamfar (2021) estimated the seismic fragility curves of typical steel buildings including SSI via the use of the BNWF model. Although these studies indicate the increased awareness of the soil-related effects, the influence of the underlying soil conditions remains vague.

The scope of the present paper is to conduct a preliminary assessment by describing and discussing the underlying damage mechanisms triggered by SSI and site amplification, influencing the seismic response of steel MRFs.

2. Methodology

To evaluate the effects of nonlinear SSI and site amplification on the seismic behavior of steel MRFs, we examine an idealized 2-story 1-bay steel MRF (Eads 2009). For this purpose, a set of ground motions is considered by explicitly modeling the underlying soil profile using the finite element method (FEM). The subsequent sections briefly describe the numerical models for both the steel MRF and soil profiles.

2.1. Examined Steel Moment Resisting Frame

Figure 1 shows the elevation view of the 2-story, 1-bay steel MRF. The steel MRF comprises wide-flange steel columns and beams with reduced beam section (RBS). The steel columns are assumed to be fixed to the foundation system with exposed column base connections, which are considered to be non-dissipative hereinafter, thereby concentrating potential inelastic deformations in the steel column base. The foundation is considered to be spread footings with no interconnecting beams. In particular, the foundation width is 2.5 m, the foundation depth equals 2.0 m, and the material is typical reinforced concrete of C30/37.

The steel MRF is modeled in the OpenSees simulation platform (McKenna 1997). Both columns and beams are idealized with elastic beam-column elements, with zero-length rotational elements at their ends (see Fig. 1). These are assigned the Modified Ibarra-Medina-Krawinkler (IMK) deterioration model (Ibarra et al. 2005; Lignos and Krawinkler 2011). This model explicitly simulates the cyclic deterioration in flexural strength and stiffness of the structural components of the steel MRF. The panel zones are modeled explicitly according to the approach summarized in Gupta and Krawinkler (1999) by featuring the trilinear panel zone model by Krawinkler (1978). A leaning column with gravity loads is linked to the steel MRF through axially rigid links (see Fig. 1) to simulate the destabilizing effects of gravity on the steel MRF response. The modeling approach is consistent with prior related studies on steel MRFs (e.g., Elkady and Lignos 2015). Damping is idealized with the Rayleigh model by considering a 2% damping ratio for the first and second vibration periods of the steel MRF.

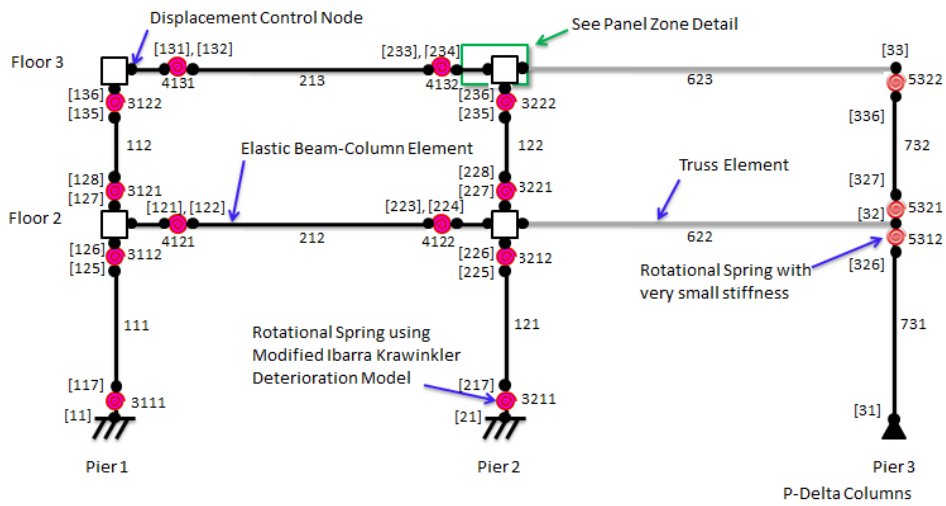

Fig. 1 - Schematic representation of the numerical model (adopted from Eads 2009)

2.2. Soil profiles examined

Seven different profiles representing four different soil types according to Eurocode 8 (CEN 2004) are numerically simulated to calculate the effect of nonlinear site amplification effects on the seismic behavior of the steel MRF. The characteristics of these profiles affect the stiffness and damping of the foundation system and are formulated as finite element columns using OpenSees.

Table 1. Earthquake recordings.

No.	**1.**	**2.**	**3.**	**4.**	**5.**	**6.**	**7.**
Vs (m/s)	>800	450	360	300	250	180	150
Type (EC8)	A	B	C	C	C	D	D

Specifically, two-dimensional "Quad" elements model a pseudo-1D soil profile. Soil nonlinearities are inherently considered using the "PressureIndependMultiYield" soil material. A single "zero-length" element is used to define the damper according to Lysmer and Kuhlemeyer (1969). Therefore, the input motion at the base of the ground column is defined in terms of velocity. The corresponding time series in terms of force is obtained by multiplying the known velocity histories by a constant factor defined as the product of the area of the base of the ground column with the specific gravity and velocity of the shear waves of the underlying rock. The size of the finite elements is determined by ensuring that enough elements "fit" in the wavelength of the earthquake under consideration. This ensures that the model is sufficiently "shredded" so that the desired propagation wave characteristics are fully considered during the nonlinear response history analysis.

Using OpenSees, each soil profile is modeled as a pseudo-1D equivalent of the physical free field to transfer the ground motion from the underlying bedrock to the surface. “PressureIndependMultiYield” material is adopted to account for nonlinear soil behavior.

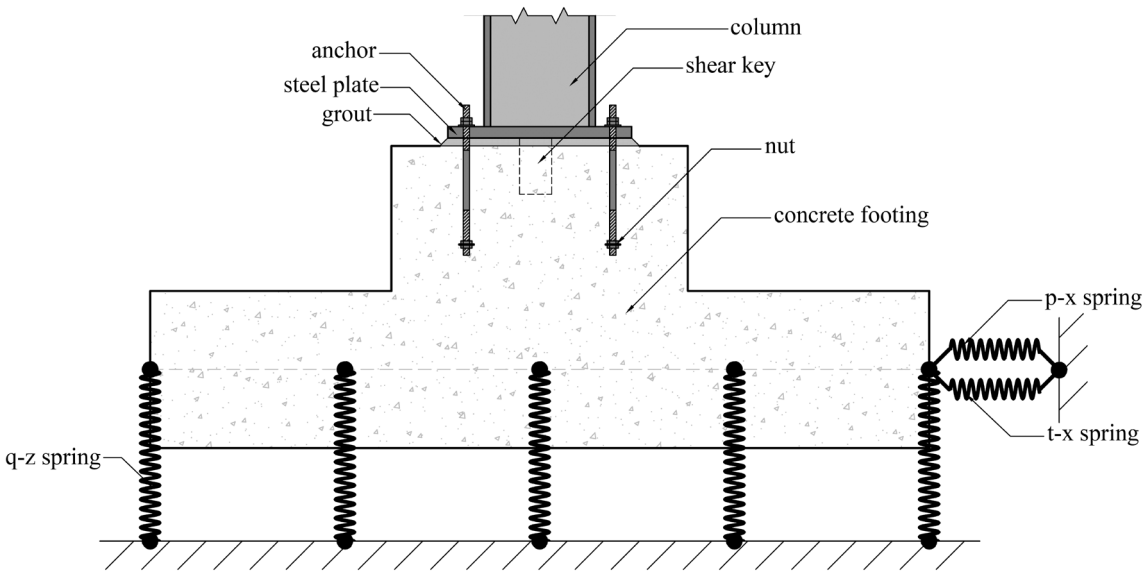

Fig. 2 - Schematic representation of the beam-on-nonlinear-Winkler-foundation model

2.3. Foundation model

The beam-on-nonlinear-Winkler-foundation (BNWF) (Harden and Hutchinson 2009) simulation of the soil-foundation response is configured using the corresponding command in OpenSees ("ShallowFoundationGen"). The foundation model consists of a system of independent nonlinear springs located at a short distance from each other as shown in Fig. 2. The vertical springs distributed along the sole are intended to simulate rocking, lifting, and settling. In contrast, the horizontal springs attached to the sides of the sole are used to simulate resistance to sliding and passive thrust. The BNWF model parameters have been calibrated to prior centrifuge experiments (Raychowdhury and Hutchinson 2009).

2.4. Ground motion records

We selected a set of eleven recorded seismic records from eleven independent seismic events. These earthquake histories were recorded on rock according to the EC8 soil type categorization. The records were selected to represent the movement on the firm bedrock, eliminating the influence and uncertainties of the soil. We deliberately excluded duplicate events to draw from a set of statistically independent records. All seismic events are characterized by $5.5 < M_w < 8.0$ and are therefore related to the Type 1 spectrum by EC8. This filtering process is rather exhaustive for existing high-traffic databases, as only a limited number of such records are available. As presented in Petridis and Pitilakis (2020), this process is followed by an iterative routine, which attempts to approach, as an average, the target range of the EC8 and further eliminates several files. Table 2 summarizes the assembled set of ground motions.

Table 2. Earthquake recordings.

No.	Location	Database Code	R_{epi} (km)	M_w	PGA (m/s^2)	$V_{s,30}$ (m/s)
1	Tabas/Iran	ESMD_59	12.00	7.35	3.16	826.00
2	Montenegro/Montenegro	ISESD_223	21.00	6.90	1.77	1083.00
3	App.Lucano/Italy	ITACA_614	9.80	5.60	1.62	1024.00
4	Kobe/Japan	NGA_1108	25.40	6.90	2.85	1043.00
5	Sierra Madre/Mexico	NGA_1645	6.46	5.61	2.71	821.69
6	Loma Prieta/USA	NGA_3548	20.35	6.93	4.12	1070.34
7	Whittier Narrows/USA	NGA_680	13.85	5.99	1.10	969.07
8	Northridge/USA	NGA_994	25.42	6.69	2.84	1015.88
9	Izmit/Turkey	T-NSMP_1109	3.40	7.60	1.65	826.11
10	East Sicily/Italy	ITACA_314	28.30	5.60	0.61	871.00
11	Western Tottori/Japan	KIK-Net_3775	31.37	6.60	1.55	967.27

2.5. Nonlinear dynamic analyses

Incremental Dynamic Analysis (IDA) is performed to estimate the seismic response of the steel frame under different levels of ground motion intensity (Vamvatsikos and Cornell 2002; Vamvatsikos and Cornell 2004). We select PGA referring to the underlying bedrock as the intensity measure (IM), and story drift ratio (SDR), accompanied by moment-rotation diagrams, to describe the structural behavior. While PGA is not an optimal IM for vulnerability studies (Kazantzi and Vamvatsikos 2015), qualifying PGA on rock conditions as an IM facilitates highlighting the differences introduced by soil nonlinearities.

3. Results

Following an extensive analysis scheme, the results are presented and evaluated in terms of both story-based (i.e., SDR) and local (i.e., moment-rotation at the beam and column ends) engineering demand parameters (EDPs). These parameters are deliberately selected, since they are frequently preferred as damage indicators in literature and common engineering practice. The labeling convention of the results is as follows:

Foundation (Soil V_s) - Earthquake ID - $PGA_{bedrock}$ [m/s^2]; where, the foundation is noted as F: for the Fixed-base, and B: for the Beam-on-Nonlinear-Winkler (SSI).

3.1 Nonlinear Static Analysis

Nonlinear static analysis of the steel MRF is conducted based a first mode lateral load pattern. Figure 3 depicts the normalized base shear, V/W versus the roof drift ratio, δ/H of the steel MRF (i.e., W: seismic weight; H is the total height of the steel MRF). The steel MRF demonstrates a full beam collapse mechanism and exhibits softening at about 3% roof drift ratio due to strength deterioration of the steel beams with RBS.

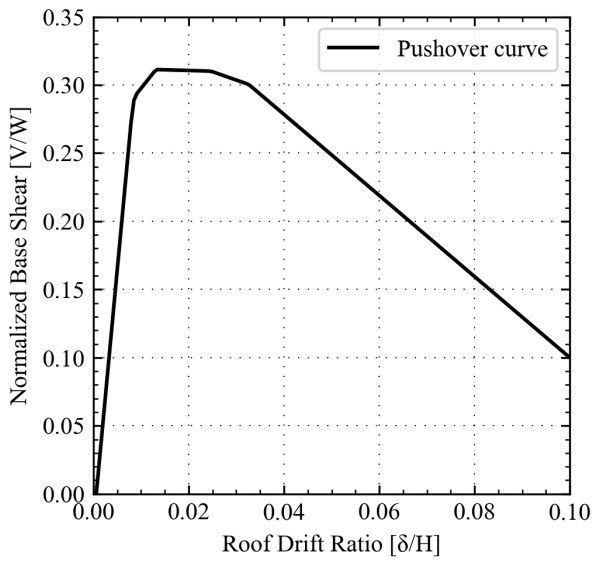

Fig. 3 - Pushover curve of the fixed-base steel MRF model

3.2 Nonlinear Dynamic Analysis

Figures 4-6 depict the SDR histories for selected scenarios, comparing the fixed-base (noted as "F") and the flexible-base (BNWF, indicated as "B") models. Since the scope of this paper is to gain an insight in behavioural characteristics of SSI effects on steel MRFs, the results are discussed with a primary focus to comprehend the 'physics' behind the observed damage mechanisms in the examined steel MRF, triggered by SSI and site effects. In the subsequent figures, SDR_1 and SDR_2 refer to the first and second SDR histories of the steel MRF, respectively.

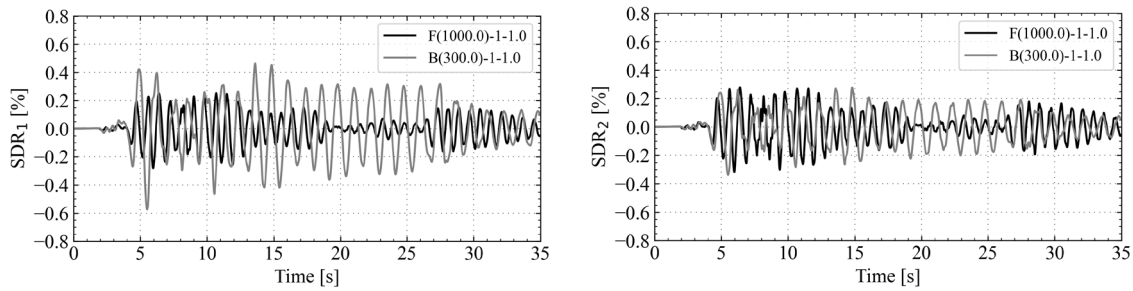

Fig. 4 - SDR for the 1st story (left) and the 2nd story (right), for the ESMD_59 record (EQ No.1 in Table 2), scaled to 1.0 m/s2 at the bedrock level (PGAbedrock), comparing a foundation fixed at the bedrock (F(1000.0)) and a BNWF on a Vs=300 m/s soil profile (B(300.0))

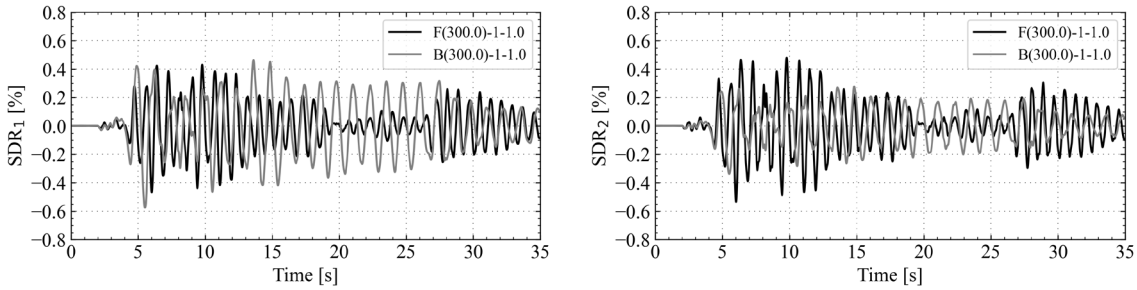

Fig. 5 – Story drift ratios for the 1st story (left) and the 2nd story (right), for the ESMD_59 record (EQ No.1 in Table 2), scaled to 1.0 m/s^2 at the bedrock level (PGAbedrock), comparing a foundation fixed (F(300.0)) and a BNWF (B(300.0)), both on a V_s=300 m/s soil profile

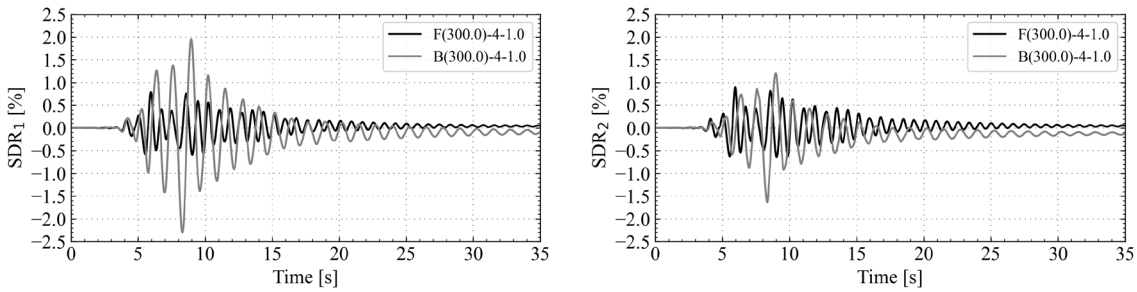

Fig. 6 - SDR for the 1st story (left) and the 2nd story (right), for the NGA_1108 record (EQ No.4 in Table 2), scaled to 1.0 m/s^2 at the bedrock level (PGAbedrock), comparing a foundation fixed (F(300.0)) and a BNWF (B(300.0)), both on a V_s=300 m/s soil profile

Referring to Figure 4, soft soil profiles tend to amplify the ground motion records at the base of the steel MRF, thereby increasing the SDR demands. However, this effect holds true for low to moderate PGA values (i.e., up to 0.3-0.4g), while for higher PGA values at bedrock level, nonlinear site effects may lead to deamplification of the ground motion record (Bazzurro and Cornell 2004). The former is characteristic of frequently occurring earthquakes (i.e., 50% probability of exceedance over 50 years). While structural damage in steel MRFs is generally negligible at about 0.6% lateral drift demands, damage to drift-sensitive non-structural elements may be a concern (e.g., Araya-Letelier et al. 2019).

Furthermore, SSI effects increase the SDR demands at the ground story level and relieve the higher stories (Figures 4-6). While the fixed-base model demonstrates a balanced SDR distribution, SSI "concentrates" the SDR maxima in the first story. This is an important observation in code-based design, since MRFs would satisfy the lateral drift requirements when idealized as fixed-base. However, in reality, SSI effects would amplify the corresponding lateral drift demands, which could be detrimental for drift-controlled steel MRFs. The same findings hold from prior work in the field by Behnamfar and Banizadeh

(2016) and Petridis and Pitilakis (2020) for reinforced concrete buildings. The present study shows that SSI effects cause residual lateral drift demands on the order of 0.25% in the 2-story steel MRF (see Figure 6) due to the increased peak SDR demands.

To provide a more elaborate perspective of the results, Figures 7 and 8 illustrate the moment-rotation diagrams for the column base (left) and the first-floor beam end (right). The simulation results suggest that in the examined cases, the flexural demands at the fixed end column reduce due to the induced soil flexibility, which is typically neglected in the conventional seismic design of steel MRFs. Conversely, the first-floor steel beams experience higher inelastic demands because of SSI, due to the shift in the moment gradient in the first-story steel columns. Interestingly, for one of the simulations shown in Fig.8, the steel beams experience softening due to the formation of inelastic local buckling within the RBS under cyclic loading.

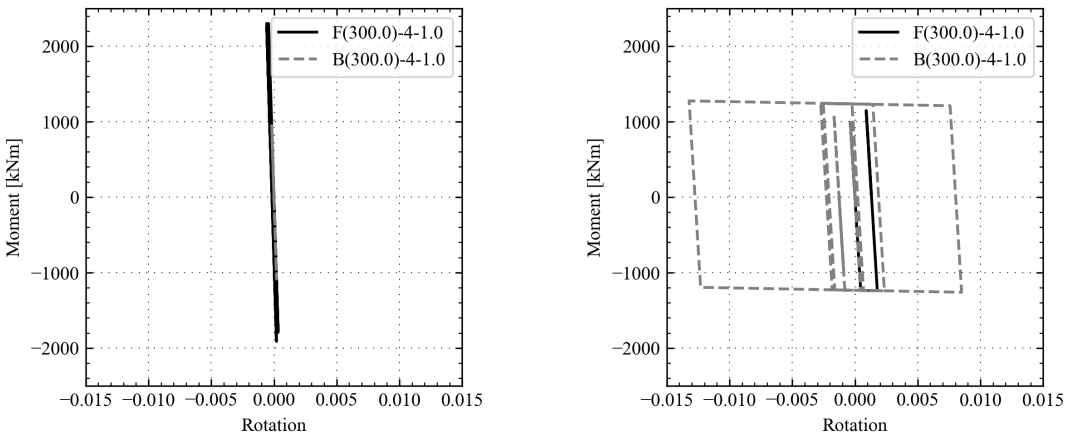

Fig. 7 – Moment rotation diagrams; column base (left) and first floor beam end (right), for the NGA_1108 record (EQ No.4 in Table 2), scaled to 1.0 m/s^2 at the bedrock level (PGAbedrock), comparing a foundation fixed (F(300.0)) and a BNWF (B(300.0)), both on a V_s=300 m/s soil profile

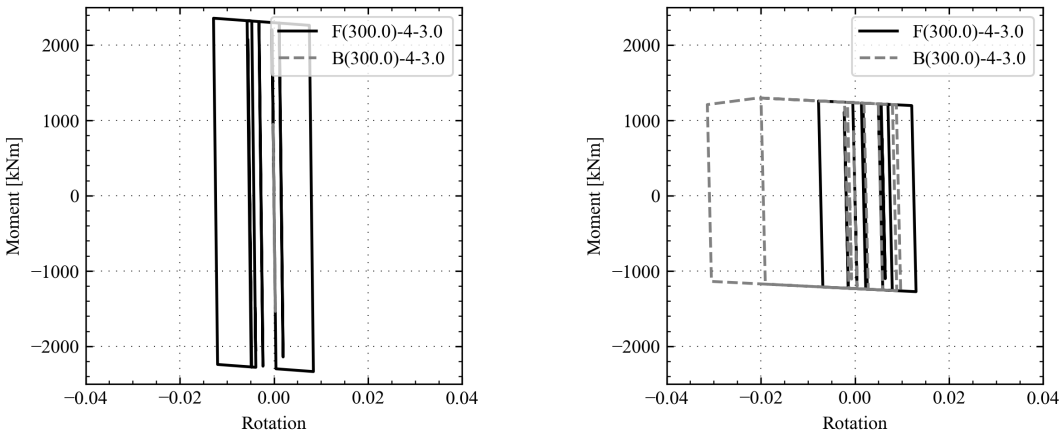

Fig. 8 - Moment rotation diagrams; column base (left) and first floor beam end (right), for the NGA_1108 record (EQ No.4 in Table 2), scaled to 3.0 m/s^2 at the bedrock level (PGAbedrock), comparing a foundation fixed (F(300.0)) and a BNWF (B(300.0)), both on a V_s=300 m/s soil profile

4. Conclusions

Aiming to introduce SSI and site amplification effects on the seismic response of steel MRFs, we examined an idealized 2-story, 1-bay steel MRF under earthquake shaking. A set of real earthquake recordings was assembled and scaled appropriately to address different intensity levels. The analyses results suggest that SSI is found to increase the

SDR demands in the first story of the examined steel MRF, whereas reduced SDR demands are observed in the second story. Moreover, the flexural demands at the fixed end column reduce due to the induced soil flexibility, accompanied by higher inelastic demands at the first-floor steel beams. Furthermore, the observed increase in terms of SDR, due to the soil flexibility, is expected to increase damage in drift-sensitive non-structural elements during frequently occurring earthquakes. To substantiate the preliminary findings further studies should be conducted.

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References

- Akhoondi, M.R., and Behnamfar, F. (2021). Seismic fragility curves of steel structures including soil-structure interaction and variation of soil parameters, Soil Dynamics and Earthquake Engineering, 143
- Araya-Letelier, G., Miranda, E., and Deierlein, G. (2019). Development and Testing of a Friction/Sliding Connection to Improve the Seismic Performance of Gypsum Partition Walls, Earthquake Spectra, 35(2): 653–677
- Bazzurro, P., and Cornell, C. (2004). Ground-Motion Amplification in Nonlinear Soil Sites with Uncertain Properties, Bulletin of The Seismological Society of America, 94: 2090-2109
- Behnamfar, F., and Banizadeh M. (2016). Effects of soil-structure interaction on distribution of seismic vulnerability in RC structures, Soil Dynamics and Earthquake Engineering, 80: 73–86.
- CEN (2004). EN 1998-1: Eurocode 8: Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings, European Committee for Standardization, Brussels
- Eads (2009). https:// opensees.berkeley.edu/ wiki/index.php/ Pushover_and_Dynamic_Analyses_of_2- Story_Moment_Frame_with_Panel_Zones_and_RBS
- El Jisr, H., Kohrangi, M., Lignos, D. (2022). Proposed nonlinear macro-model for seismic risk assessment of composite-steel moment resisting frames, Earthquake Engng Struct Dyn., 51: 1180–1200
- Elkady, A., and Lignos, D. G. (2015). Effect of gravity framing on the overstrength and collapse capacity of steel frame buildings with perimeter special moment frames, Earthquake Engng Struct. Dyn., 44, 1289– 1307
- Flores, F.X., Charney, F., Lopez-Garcia, D. (2014). Influence of the gravity framing system on the collapse performance of special steel moment frames, Journal of Constructional Steel Research, 101: 351–362
- Gupta A., and Krawinkler H. (1999). Seismic Demands for Performance Evaluation of Steel Moment Resisting Frame Structures, Technical Report 132, The John A. Blume Earthquake Engineering Research Center, Department of Civil Engineering, Stanford University, Stanford, CA
- Harden, C. W., and Hutchinson, T. C. (2009). Beam-on-Nonlinear-Winkler-Foundation Modeling of Shallow, Rocking-Dominated Footings, Earthquake Spectra, 25(2), 277–300
- Ibarra, L. F., and Krawinkler, H. (2005). “Global collapse of frame structures under seismic excitations,” Technical Report 152, The John A. Blume Earthquake Engineering Research Center, Department of Civil Engineering, Stanford University, Stanford, CA
- Ibarra, L. F., Medina, R. A., and Krawinkler, H. (2005). “Hysteretic models that incorporate strength and stiffness deterioration,” Earthquake Engineering and Structural Dynamics, Vol. 34, 12, pp. 1489-1511
- Kazantzi, A. K., and Vamvatsikos, D. (2015). Intensity measure selection for vulnerability studies of building classes, Earthquake Engng Struct. Dyn., 44: 2677– 2694
- Krawinkler, H. (1978). Shear in beam Shear in beam-column joints in seismic design of steel frames, Engineering Journal, 15: 82-91
- Lignos, D. G., and Krawinkler, H. (2009). “Sidesway Collapse of Deteriorating Structural Systems under Seismic Excitations,” Technical Report 172, The John A. Blume Earthquake Engineering Research Center, Department of Civil Engineering, Stanford University, Stanford, CA

- Lignos, D. G., and Krawinkler, H. (2011). “Deterioration Modeling of Steel Components in Support of Collapse Prediction of Steel Moment Frames under Earthquake Loading", ASCE, Journal of Structural Engineering, Vol. 137 (11), 1291-1302
- Lysmer J., Kuhlemeyer A.M. (1969). Finite dynamic model for infinite media, Journal of the Engineering Mechanics Division, ASCE, 95: 859-877
- Mashhadi, S., Asadi, A., Homaei, F., Tajammolian H. (2021). Seismic response of mid-rise steel MRFs: the role of geometrical irregularity, frequency components of near-fault records, and soil-structure interaction, Bull Earthquake Eng 19, 3571–3595
- McKenna, F. (1997) Object oriented finite element analysis: Frameworks for analysis algorithms and parallel computing, Ph.D. dissertation, University of California, Berkeley.
- Mylonakis, G., Gazetas, G. (2000). Seismic soil-structure interaction: Beneficial or detrimental?, Journal of Earthquake Engineering, 4 (3): 277-301
- Petridis, C., and Pitilakis, D. (2020). Fragility curve modifiers for reinforced concrete dual buildings, including nonlinear site effects and soil-structure interaction. Earthquake Spectra 36(4), 1930–1951
- Petridis, C., and Pitilakis, D. (2021). Large-scale seismic risk assessment integrating nonlinear soil behavior and soil–structure interaction effects. Bulletin of Earthquake Engineering, 19, 6423–6441
- Pitilakis, K., Karapetrou, S., Fotopoulou, S. (2013). Consideration of aging and SSI effects on seismic vulnerability assessment of RC buildings, Bulletin of Earthquake Engineering, 12: 17551776
- Rajeev, P., and Tesfamariam, S. (2012). Seismic fragilities of non-ductile reinforced concrete frames with consideration of soil structure interaction, Soil Dynamics and Earthquake Engineering, 40: 78–86
- Raychowdhury P., Hutchinson T.C. (2009). Performance evaluation of a nonlinear Winkler-based shallow foundation model using centrifuge test results, Earthquake Engineering and Structural Dynamics, 38 (5): 679-698
- Vamvatsikos, D. and Cornell, C.A. (2004). Applied incremental dynamic analysis, Earthquake Spectra, 20 (2): 523-553
- Vamvatsikos, D., and Cornell, C.A. (2002). Incremental dynamic analysis, Earthquake Engineering & Structural Dynamics, 31(3): 491-514