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Preliminary study of a seismic-resilient steel pilot building equipped with low-damage connections

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Abstract

According to current seismic design codes, structures are designed to exhibit an elastic behaviour or slight damage in case of frequent (low intensity) seismic events. Conversely, in the case of rare (high intensity) seismic events, more widespread damage is allowed. In fact, according to the latter case, structures are typically designed to concentrate the seismic damage into dissipative fuses, whose ductility and energy dissipation capacity is properly designed through the adoption of specific detailing rules. This approach allows the achievement of the safety requirements, with considerable damage to the structural components and large residual drifts, which can significantly compromise the building's reparability. To overcome these drawbacks, recent efforts are aimed at developing innovative seismic resilient structures to reduce structural damage and repair time. Among others, steel Moment Resisting Frames equipped with friction devices in beam-to-column joints have emerged as a promising and effective solution that simultaneously ensures the seismic input energy dissipation capacity and the damage-free behaviour of the system. In this direction, relevant research studies have been carried out within the RFCS-FREEDAM research project, demonstrating the high potential of friction joints in drastically reducing the seismic damage of steel structures. Within this context, the free-from-damage technology developed within the FREEDAM research project is going to be implemented in a demonstration building to be erected at the University Campus of Salerno. The present paper illustrates the preliminary design and results of the numerical simulations in OPENSEES. Non-linear static analyses and Incremental Dynamic Analyses are performed to obtain the engineering demand parameters of interest while accounting for the record-to-record variability.

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1. Introduction

According to the conventional seismic design philosophy, suggested by most current codes and guidelines (*e.g.,* Eurocode 8, 2005), structures are conceived to concentrate the seismic damage into specific dissipative zones able to provide high ductility and energy dissipation capacity in case of 'rare' (high intensity) seismic events (*i.e.,* Ultimate Limit State). Within steel Moment Resisting Frames (MRFs), this strategy consists in adopting over-strengthened columns and full-strength connections, concentrating the damage at the beams' ends (Mazzolani and Piluso 1997; Bruneau et al. 1998). However, such an approach implies extensive difficult-to-repair damage, often distributed throughout many non-replaceable structural elements and residual drifts, hence leading to large direct (*e.g.,* casualties, repair cost) and indirect (*e.g.,* downtime) losses which are not acceptable from both the economic and social perspectives (McCormick et al. 2008; Freddi et al. 2019).

To overcome these deficiencies, several research studies, as well as practical applications, are currently focusing on the development of innovative low-damage systems, chasing the objectives of enhancing their seismic performance and structural resilience (Chancellor et al. 2014; Freddi et al. 2021; Fang et al. 2022). Many research works on steel MRFs have been focused on replacing the conventional full-strength beam-to-column connections with dissipative partial-strength joints, whose main feature is the ability to provide lower strength than the connected members, thanks to the proper design of weak nodal components. An example of such an approach is represented by the so-called Sliding Hinge Joint (SHJ), conceived by Butterworth and Clifton (2000), relying on the intuition of Yang and Popov (1995). It is based on the inclusion of Friction Devices (FDs) in beam-to-column connections to dissipate the seismic energy with negligible damage. The results of these studies demonstrated the advantages of damage-free systems in terms of easy replaceability and high dissipation capacity and promoted many successive studies in this direction.

Recently, within the framework of the RFCS-FREEDAM (FREE from DAMage) European research project, several research studies have been carried out on beam-to-column connections equipped with FDs. The damper typology included in this connection was extensively studied in previous experimental works, which have addressed significant aspects, such as the response of the FDs under cyclic loading histories and the behaviour of pre-loadable bolts at installation and over their service-life (*e.g.,* Cavallaro et al. 2017; Latour et al. 2018a). Different configurations of symmetric removable FDs for low-damage beam-to-column joints have been tested and investigated, observing a satisfactory overall performance with a stable and predictable hysteretic response controlled by adequately regulating the tightening torque of pre-loadable high-strength bolts (*e.g.,* Latour et al. 2015 and 2018b; Piluso 2018). The FREEDAM research project performed an in-depth investigation of the behaviour of friction joints, including a broad set of experimental tests, extensive Finite Element (FE) model, simulations and analytical work, and provided design rules and standardised components (Francavilla et al. 2020; Tartaglia et al. 2021; Di Benedetto et al. 2021). The results demonstrated the high potential of friction beam-to-column joints in minimising structural damage, hence guaranteeing a fast and cheap reparability of the structure even in the aftermath of severe seismic events. It is worth highlighting that, although the investigated low-damage connections are able to guarantee a high energy dissipation capacity and a free-from-damage behaviour, they are not endowed with self-centring capability. To solve this problem, advanced technologies based on the use of additional cables or post-tensioned bars are already available (Elettore et al. 2021a 2021b.)

Within the RFCS-DREAMERS project, the free-from-damage technology developed during the FREEDAM project is going to be implemented in a demonstration building erected at the University Campus of Salerno, representing a step forward in the available technologies for seismic protection of steel buildings in Europe. The main objective of the projects are: *i)* to promote the use of resilient and sustainable steel structures in earthquake-prone countries; *ii)* to raise awareness about the competitiveness of a damage-free building and to increase performance levels; *iii)* to protect people from the disruption deriving from the interruption of building's functionality after severe seismic events; *iv)* to demonstrate the ease of construction and the benefits deriving from the application of innovative damage-free joints. The present paper illustrates the preliminary design and the results of the numerical simulations in OPENSEES of the DREAMERS project. Non-linear static analyses are performed to assess both the global seismic behaviour of the building and the local response of the friction connections. Furthermore, Incremental Dynamic

Analyses (IDAs) (Vamvatsikos et al. 2002) are performed to evaluate the dynamic response of the structures accounting for the record-to-record variability.

2. Design of the building and the connections

The pilot building is going to be constructed at the University of Salerno on a plot located in the northern part of the Campus of Fisciano (Salerno, Italy). The building has plan dimensions of approximately 25 m × 15 m and an overall height of about 12 m. The main entrance is placed at the ground floor level. The first and second floors are dedicated respectively to a medical laboratory and offices. The stairwell is conceived to be structurally independent of the main body of the pilot building, and it is located on the east side. Fig. 1(a) shows the plan view of the building, which has three storeys, five bays in the x-direction and three bays in the y-direction. Two seismic resistant-perimeter MRFs are placed in both the main directions, while the interior part is composed of gravity frames. The floor slabs use the ArcelorMittal slim floor system with the COFRADAL 260 steel-concrete composite solution. The floor beams are made with HEB300 and HEB240 cut-off beams according to the scheme depicted in Fig. 1(a). Loads and masses applied to the structure are summarised in Table 1. The design is performed according to Eurocode 8 (2005) and the Italian Code (NTC, 2018). For brevity, this study deals only with the seismic performance of the MRFs in the y-direction, whose elevation view is shown in Fig. 1(b).

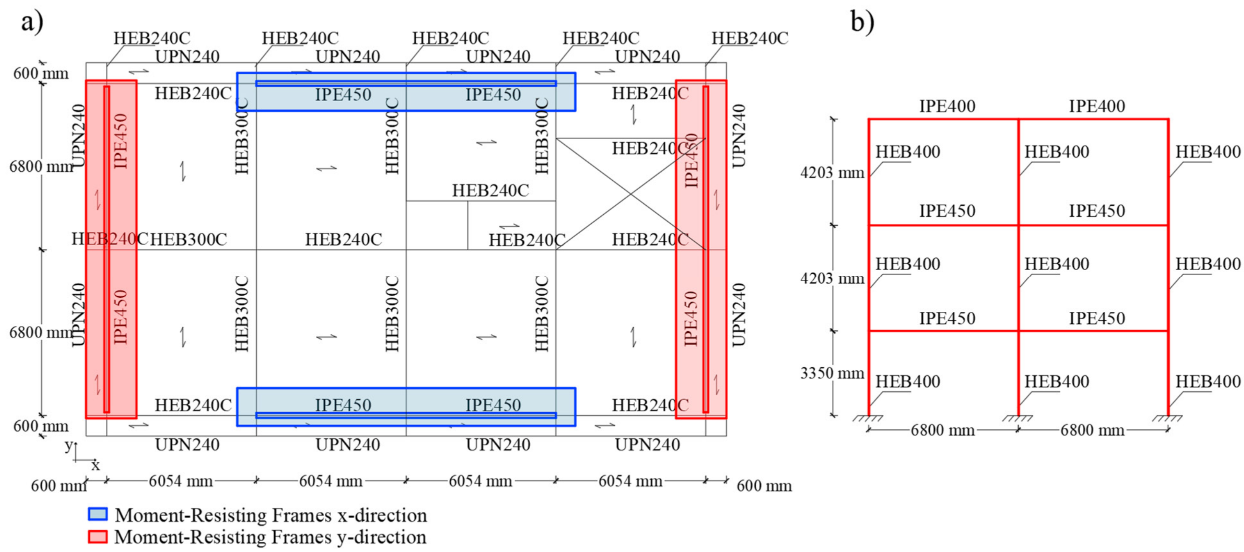


Fig. 1 – Plan view (a) and Elevation view (b) of the building

Table 1. Dead loads and masses

	First level	Second level	Third level
Dead load (kN/m^2)	5.75	5.75	4.55
Claddings (kN/m)	4.41	4.41	-
Live load (kN/m^2)	3.00	3.00	-
Mass (tons)	305.00	305.00	210.00

The Type 1 elastic response spectrum with a Peak Ground Acceleration (*i.e.,* PGA) equal to 0.178g, soil type B and class III (*i.e.,* $C_U = 1.5$) is considered for the definition of the Ultimate Limit State (*i.e.,* ULS, $T_R = 712$). The behaviour factor (q) used for the definition of the design spectrum is assumed to equal 6.5 according to the requirements of the Italian Code (NTC, 2018) for MRFs in DCH. Additionally, as suggested by the Italian Code (NTC, 2018), the resistance checks are performed also for the OLS (*i.e.,* Operational Limit State) by considering a behaviour factor q = 1 and for the DLS (*i.e.,* Damage Limit State) by considering q = 1.5. Beams are IPE 450 for the first two levels and IPE 400 at the roof level, while all columns are HEB400. All the structural elements are made of S355JR-type steel.

The interstorey drift limit for the DSL requirements is assumed as 0.5%, as suggested by the Eurocode 8 (2005). Fig. 2 shows the vertical configuration (*i.e.,* VFC-configuration) of the FREEDAM beam-to-column joint in which the friction damper's plan is parallel to the beam's web. The joint is constituted by a rib bolted to the lower beam flange and two L-stubs bolted to the rib and the column's flange. The friction pads, made of steel plates coated with thermally sprayed material, are located between the L-stub and the rib and pre-stressed with pre-loadable high-strength bolts. The top beam flange is connected to the column flange with a bolted T-stub, fixing the Centre of Rotation. Additional information regarding this joint typology is provided in Francavilla et al. (2020) and Tartaglia et al. (2021). The main properties of the adopted devices are reported in Table 2.

In addition, the design is also performed and checked by following the Theory of Plastic Mechanism Control (TPMC) proposed by Mazzolani and Piluso (1997) to assure the development of a global failure mode. The interested reader can refer to Montuori et al. (2015) and Nastri et al. (2019).

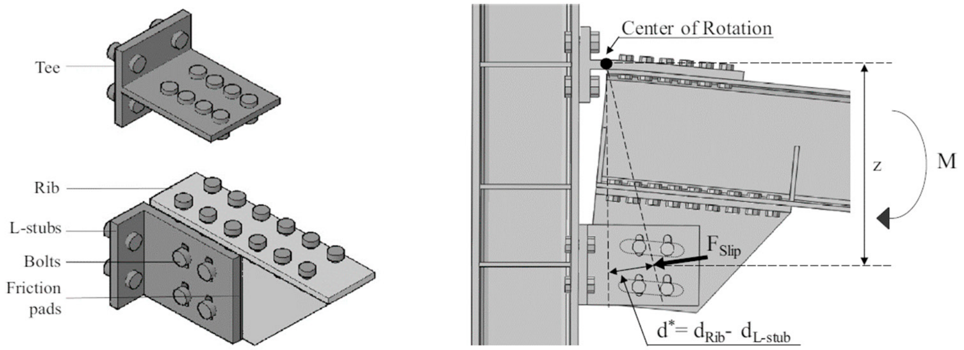


Fig. 2 – FREEDAM connection (adapted from Tartaglia et al., 2021)

Table 2. Device Properties

	First level	Second level	Third level
MARK	FREEDAM – IPE 450/0.4	FREEDAM – IPE 450/0.4	FREEDAM – IPE 400/0.3
Name	D-2A	D-2A	D1
$F_{slip,Rd}$ [kN]	345.3	345.3	244.2
$M_{j,Rd}$ [kNm]	242	242	139
Bolts	M20 HV 10.9	M20 HV 10.9	M16 HV 10.9
Number of bolts, n_b	4	4	4
Number of surfaces, n_s	2	2	2
Preload force, $F_{p,d}$ [kN]	93.64	93.64	66.23

3. Numerical modelling

A 2D non-linear FE model of the structure is developed in OPENSEES (Mazzoni et al. 2009). Beams are modelled by a lumped plasticity approach where the internal part of the beams is modelled with '*elastic beam-column elements*'. Conversely, columns are modelled by a distributed plasticity approach using '*non-linear beam-column elements*'. The section aggregator function in OPENSEES (Mazzoni et al. 2009) accounts for the column's shear stiffness. Beams and columns are modelled using the '*Steel01*' material (Mazzoni et al. 2009) with 355 MPa yield strength and 0.2% post-yield stiffness ratio. The beam-to-column joint modelling strategy is consistent with Di Benedetto et al. (2021). The rigid elements of the joints are modelled with '*elastic beam-column elements*' (Mazzoni et al. 2009). The FD is modelled by a '*zero-length element*' characterised by '*uniaxial hysteretic material*' with symmetric trilinear force-displacement law. This material adopts a yielding force equal to the sliding force and very low post-elastic hardening. In addition, geometric nonlinearities are considered in the elements of the frame. The P-Δ effects related to the displacement and the axial forces in the gravity columns are considered with an additional leaning column, modelled consistently with the strategy proposed by Ahmadi et al. (2018). Additional information regarding the modelling strategy is reported in Elettore et al. (2022).

4. Pushover analysis

Non-linear static analyses with a distribution of lateral forces defined according to the first mode have been performed on the MRF. The results are shown in Fig. 3(a), illustrating the MRF's base shear versus the top storey displacement. It is worth stressing that, thanks to the design procedure, the structures are characterised by a homogeneous inelastic demand at all storeys. In addition, it is worth noting that, thanks to the TPMC design procedure, the global failure mode mechanism curve characterises the collapse condition of the structure (Mazzolani and Piluso 1997). The pushover curve also provides preliminary information on the activation hierarchy of the different mechanisms within the structure. Fig. 3(b) shows the moment-rotation curve of a FREEDAM joint placed at the second storey including the indication of the points corresponding to the different limit states. As expected, the FREEDAM connections do not activate at the OLS, while they start sliding at the DLS, as expected from the design procedure.

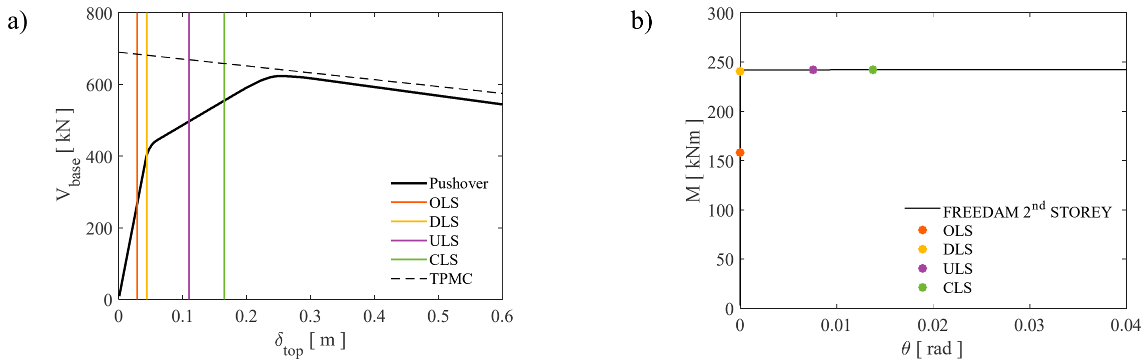
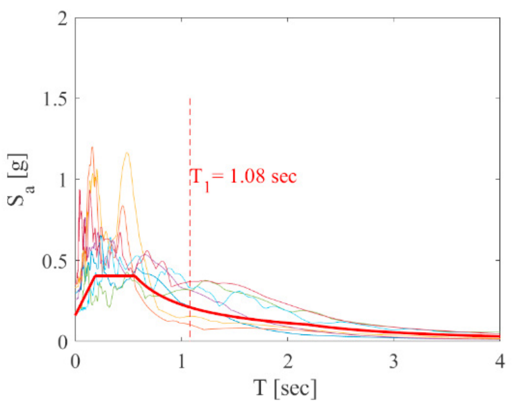


Fig. 3 – Pushover analysis results: (a) Pushover curve; (b) FREEDAM connection for the second storey

5. Incremental Dynamic Analysis (IDAs)

IDAs (Vamvatsikos et al. 2002) are performed by considering a suite of ground motion records, scaled to increasing Intensity Measure (IM) values to cover the range from elastic to a large non-linear seismic response of the frame. The spectral acceleration corresponding to the first vibration mode ($Sa(T_1)$) is used as IM where T_1 = 1.08 sec. A set of 7 natural ground motions records is selected from the ITALIAN Database (Iervolino et al. 2010) with the parameters summarized in Fig. 4. The records have been selected such that their mean elastic spectrum is kept between 90% and 130% of the design spectrum.



Selection Parameters

Moment magnitude M_w ranging from 5 to 7

Epicentral distance R < 30 km

Site class B

Spectrum-compatibility in the range 0.2 T_1 and 2 T_1

Fig. 4 – Selected ground motions records

Global Engineering Demand Parameters (EDPs) [*i.e.,* peak ($\theta_{s\text{-}max}$) and residual interstorey drifts ($\theta_{s\text{-}res}$)] are recorded to investigate the frame's seismic response while accounting for the influence of the record-to-record variability. Highlighted in the figures are the mean curves among all ground motions as a synthesis of the demand values for both quantities, while the single IDA curves are shown in grey. Fig. 5(a), (b) and (c) show the sample of the demand for the peak interstorey drifts $\theta_{s\text{-}max}$ vs the IM for the first, second and third storeys, respectively. Fig. 5(d), (e) and (f) show the residual interstorey drifts $\theta_{s\text{-}res}$ vs the IM for all the storeys. To evaluate the self-centring capacity of the building, two limits are considered: the drift limit of 0.5%, which is conventionally associated with building reparability (McCormick et al. 2008) and the limit of 0.2%, which ensures that no structural realignment is necessary (FEMA P58-1). It is possible to observe that the frame experiences mean residual interstorey drifts lower than $\theta_{s\text{-}res\text{-}Limit}$ = 0.5% at the ULS and the CLS for all storeys. Conversely, the limit $\theta_{s\text{-}res\text{-}Limit}$ = 0.2% is not always satisfied.

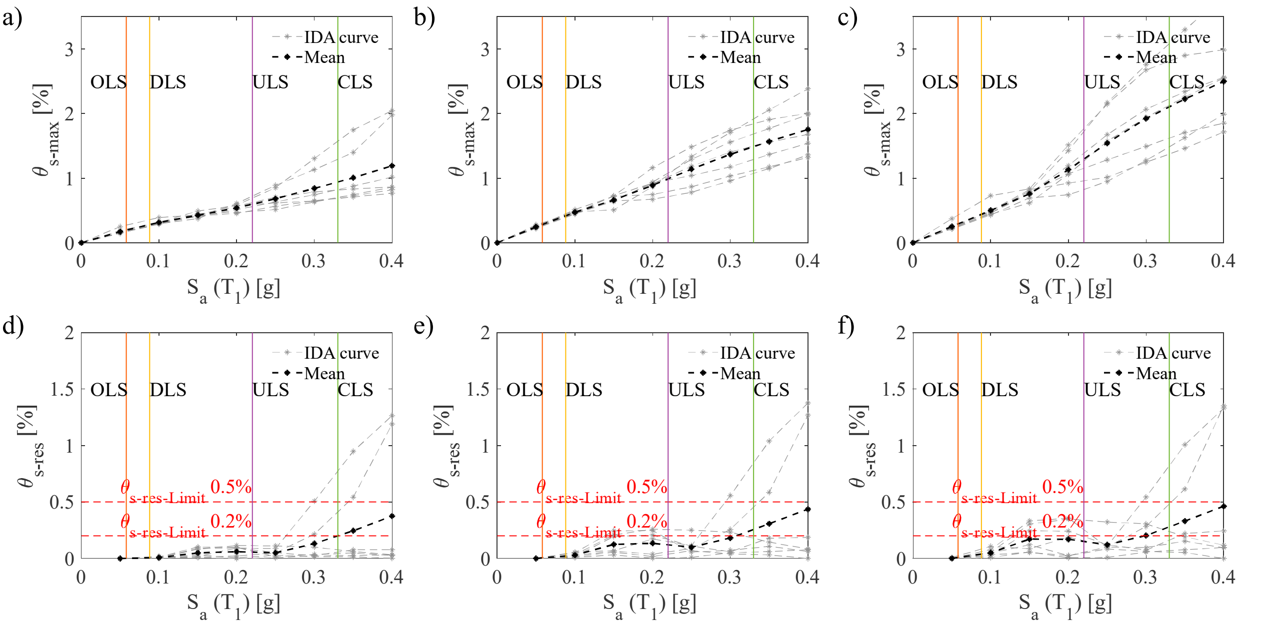
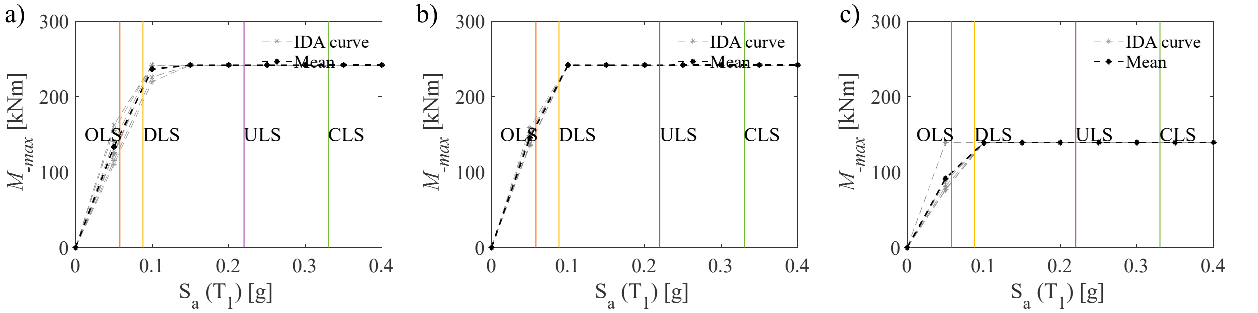


Fig. 5 – Global Engineering Demand Parameters (EDPs): Peak interstorey drifts for the (a) First; (b) Second; and (c) Third storey. Residual interstorey drifts for the (d) First; (e) Second; and (f) Third storey.

Additionally, the local behaviour is also investigated by monitoring two local EDPs (*i.e.,* the maximum moment - M_{max} - and the maximum rotation - θ_{max} - experienced by the FREEDAM connections). Fig. 6(a), (b) and (c) show the maximum moment experienced by the connections placed at the first, second and third storey, respectively. As expected from the design, the sliding of the connection is achieved after the DLS. This behaviour is confirmed by observing the maximum rotation (*i.e.,* θ_{max}) experienced by the connections, shown in Fig. 6(d), (e) and (f).



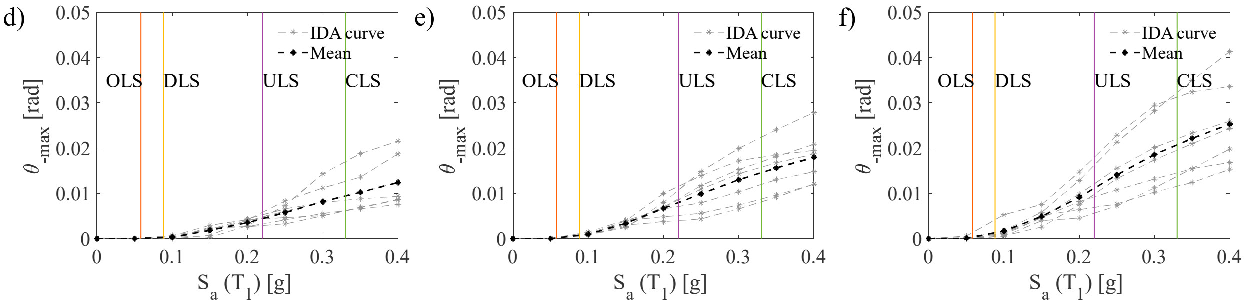


Fig. 6 – Local Engineering Demand Parameters (EDPs): Max moment achieved in the FREEDAM connection for the (a) First; (b) Second; and (c) Third storey. Max rotation in the FREEDAM connection for the (a) First; (b) Second; and (c) Third storey

In addition, for one single ground motion record, Fig. 7 shows the moment-curvature hysteretic behaviour of the bottom sections of the first storey columns of the structure at the CLS intensity. It is observed that the column bases experiences large plastic deformations and damage, thus leading to the need for repair measures after strong earthquakes under both the USL and the CLS. Therefore, the possibility of inserting a damage-free self-centring system localised in the column bases is also evaluated, including in the structure a system that facilitates the re-centring as well as the protection of the first storey columns from yielding.

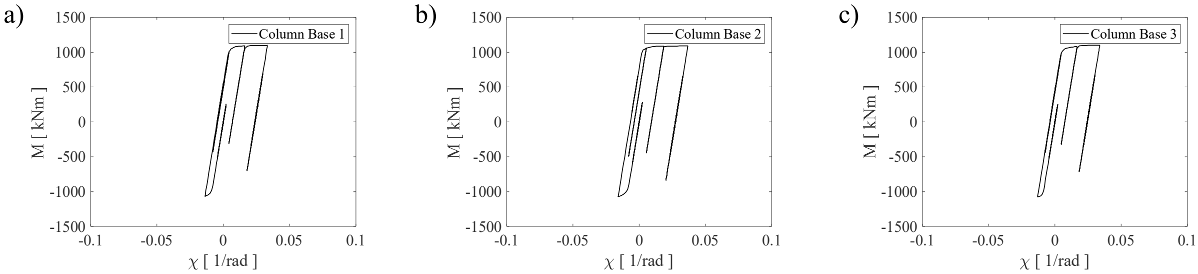


Fig. 7 - Moment-curvature relationship in the bottom section of the three first storey columns for a single record scaled at the CLS

6. Conclusions

The present paper provides preliminary information on the design and numerical simulations in OPENSEES of a seismic-resilient steel pilot building equipped with the free-from-damage FREEDAM connections. Non-linear static analyses have been performed to assess both the global behaviour of the building and the local response of the connections. Incremental Dynamic Analyses (IDAs) are carried out to evaluate the dynamic response of the structure accounting for the record-to-record variability. Global and local Engineering Demand Parameters (EDPs) (*i.e.,* peak and residual interstorey drifts, maximum moment and rotation experienced by the connections) are recorded to investigate the frame's seismic response. Some preliminary conclusions can be drawn: *i)* Results from the non-linear static and dynamic analyses show the adequacy of the design assumptions and confirm the consistency of the design methodology; *ii)* Results from the non-linear static analyses provide preliminary information on the activation hierarchy of the different mechanisms within the structure; *iii)* Results from the Incremental Dynamic Analysis show that the frame experiences residual interstorey drifts lower than the limit of 0.5% for all the storeys under the seismic intensities of interest, in terms of mean response. Conversely, the limit of 0.2% is not always satisfied; *iv)* As expected from the design procedure, The FREEDAM connections do not activate at the Operational Limit State and they start sliding at the Damage Limit State; *v)* The column bases experience large plastic deformations and damage, thus leading to the need for repair measures after strong earthquakes. Therefore, aiming at achieving an adequate structural resilience, the possibility of inserting a damage-free self-centring system localised in the column bases is also evaluated, including in the structure a system that facilitates the re-centring, as well as the protection of the first storey columns from yielding.

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