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Research paper

Performance of seismic shear panels under near-field motions

Mariella Diaferio *

Polytechnic University of Bari, Department of Sciences in Civil Engineering and Architecture
**Corresponding author E-mail: mariella.diaferio@poliba.it*

Abstract

The purpose of this study is to describe the structural response of steel frames with different stiffness and geometrical characteristics subjected to near-field earthquakes. Such actions, in fact, may be responsible of high levels of damage, as they are characterized by pulses of high amplitude. Thus, the effects on the structures are quite different from the ones induced by far-field earthquakes. In detail, the main aim is to evaluate the performance of such frames equipped with passive devices: two hysteretic dampers are considered which during the seismic action are mainly interested by shear stresses. Nevertheless, the main characteristics of their hysteretic behavior are different and, consequently, for a correct use the comparison of the performance is an important tool. Thus, the response of the protected frames is evaluated varying the characteristics of the near-field motions.

Keywords: *Near-Field Earthquakes, Seismic Passive Devices, Performance Parameters.*

1. Introduction

In literature it has been many times underlined the differences of the characteristics of the seismic action between far- field and near -field areas. In these last cases [1]-[5] the motion is characterized by short duration pulses and by high amplitude peak ground velocities and accelerations. In fact, in the area surrounding an active fault surface system, and for a distance from the fault surface projection equal nearly to linear fault dimension, the waves are strongly influenced by the source. In particular, near-field ground motions exhibit one or more dominant pulses at the beginning of the seismogram mostly oriented in the fault-normal direction, high vertical acceleration, and the acceleration response spectra show a distinct low pulse period. Thus, the structures may show worse performance than the ones under far-field earthquakes.
In order to increase the safety level of a structure, many devices have been proposed in literature.
However, due to their own characteristics, the performance under near-field motions may vary. In the present study two different hysteretic dampers are considered and their performance are investigated with respect to some ad hoc chosen parameters of the structural response.
Moreover, also the characteristics of the seismic action are varied to perform a wide analysis which makes use of numerous near field accelerograms. In detail, the present study analyses the response of steel moment-resisting buildings subjected to 21 near field earthquakes.

2. The near-field earthquakes

The 21 different near-field earthquakes examined in the present study are summarized in Table 1, while in figure 1 the correspondence acceleration response spectra are shown.
In the subsequent analysis, the values of their peak acceleration have been normalized at 0.35 g, which corresponds to high seismic input

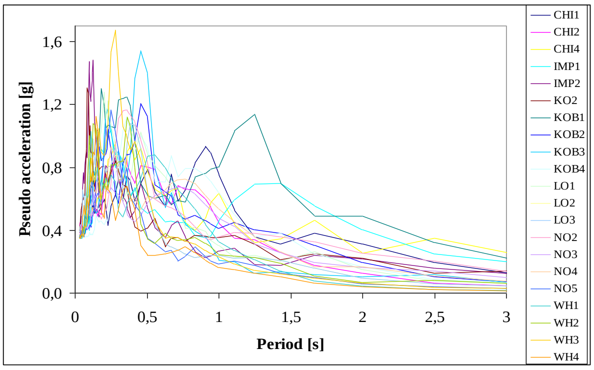

Fig. 1: Pseudo acceleration spectrum. Records are taken from PEER (Pacific Earthquake Engineering Research) database (damping equal to 5%) [6].

In figure 2 the relationship between the magnitude and the closest distance of all the considered near field records is plotted.

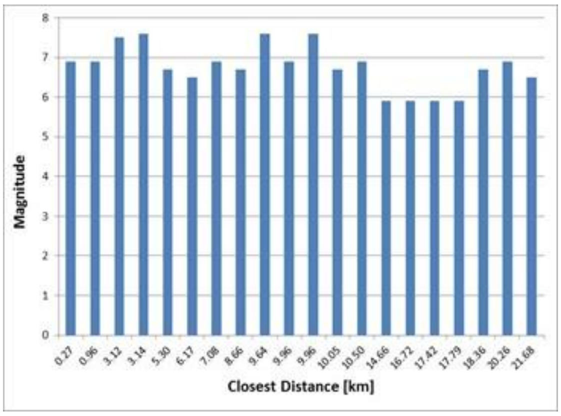

Fig. 2: Distribution magnitude *vs.* closest distance for all the considered near field records [see Table 1]

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Table 1: Examined near-field records (PEER database [6])

Event	ID	Station	Soil type	PGA [g]	PGV [cm/s]	PGD [cm]	Magnitude	Epicentral distance [km]	Closest distance [km]
Imperial Valley 15/10/79	IMP1	El Centro Array #10	Deep broad soil	0.224	41	19.38	6.5	26.3	6.17
Imperial Valley 15/10/79	IMP2	El Centro Array #1	Deep broad soil	0.134	16	9.96	6.5	35.18	21.68
Whittier 1/10/87	WH1	Bell Gardens- Jaboneria	Deep broad soil	0.219	18.9	2.54	5.9	11.77	17.79
Whittier 1/10/87	WH2	Baldwin Park- Holly Ave	Deep broad soil	0.127	8.6	2.5	5.9	11.36	16.72
Whittier 1/10/87	WH3	Arcadia - Campus drive	Deep broad soil	0.3	21	3.12	5.9	9.89	17.42
Whittier 1/10/87	WH4	Alhambra- Fremont sch.	Deep broad soil	0.414	16.3	2.32	5.9	6.77	14.66
Loma Prieta 18/10/89	LO1	Anderson Dam	Deep broad soil	0.244	20.3	7.73	6.9	26.57	20.26
Loma Prieta 18/10/89	LO2	Gilroy- Gavilan Coll.	Stiff soil	0.357	28.6	6.35	6.9	28.98	9.96
Loma Prieta 18/10/89	LO3	Gilroy Array #1	Rock	0.473	33.9	8.03	6.9	11.2	10.5
Northridge 17/10/94	NO2	Sylamr- Olive View Med FF	Deep broad soil	0.843	129.6	32.68	6.7	16.77	5.3
Northridge 17/10/94	NO3	Sun Valley - Roscoe Blvd	Deep broad soil	0.443	38.2	10.04	6.7	12.35	10.05
Northridge 17/10/94	NO4	Arleta - Nordhoff Fire Sta	Deep broad soil	0.344	40.6	15.04	6.7	11.1	8.66
Northridge 17/10/94	NO5	Beverly Hills- 12520 Mulhol	Deep narrow soil	0.617	40.8	8.57	6.7	16.27	18.36
Kobe 16/1/95	KOB2	Takarazuka	Soft soil	0.694	85.3	16.75	6.9	38.6	0.27
Kobe 16/1/95	KOB3	Nishi- Akashi	Soft soil	0.509	37.3	9.52	6.9	8.7	7.08
Kobe 16/1/95	KOB4	KJMA	Soft soil	0.821	81.3	17.68	6.9	18.27	0.96
Kocaeli Turkey 17/8/99	KO2	Sakarya	Deep broad soil	0.376	79.5	70.52	7.5	33.24	3.12
Chi- Chi Taiwan	CHI1	CHY101	Deep broad soil	0.44	115	68.75	7.6	31.96	9.96
Chi- Chi Taiwan	CHI2	CHY028	Deep broad soil	0.821	67	23.28	7.6	32.67	3.14
Chi- Chi Taiwan	CHI4	CHY024	Deep broad soil	0.278	52.9	43.62	7.6	24.1	9.64

As it can be observed, the records can be considered representative of earthquakes with magnitude varying in the range 5.9-7.6 and closest distance in the range 0-22 km. All the considered records have been implemented considering both the horizontal and vertical components of the motion.

3. The moment resisting frames

In order to perform a wide analysis also the mechanical characteristics of the structure have been varied, consequently six different steel frames have been examined.
The adopted nomenclature for the steel frames is: CxPy where C stays for bays, thus x is the number of bays of the frame, while P stays for floors consequently y is the number of floors which compose the steel frame. Moreover, the presence of R at the beginning of the frame identification name denotes the use of columns sections with a higher stiffness than the correspondence frame without "R". The R frames have been considered to vary the natural frequencies of the structure.
Furthermore, the frames are characterized by bays 6 m long, however the interstorey height is equal to 4m at the first level and to 3 m at the higher levels.
In figure 3 are summarized the sections adopted for all the structural elements which compose each steel frame, while in table 2 are reported the first three natural periods of the examined steel frames.

Table 2: First three natural periods of the examined steel frames

ID Frame	I mode [s]	II mode [s]	III mode [s]
RC4P1	0,28	0,071	0,04
C4P1	1,029	0,072	0,054
C2P3	1,45	0,46	0,277
RC2P6	1,47	0,43	0,2
C2P6	2,04	0,75	0,41
C2P9	2,54	0,9	0,527

4. The passive dissipation devices

In order to improve the safety level of such structures, the responses of the aforementioned moment- resisting frames have been evaluated also considering the installation of two different passive energy dissipation systems to describe the performances of such solution. The dissipation devices are considered installed at each level of the frames and in only one bay. The devices are positioned between the joint of two diagonal members (HEB profiles) and the upper floor beam.
The two different passive devices analysed in the study react in their main plane and are subjected to shear forces. In the following the main characteristics of the passive dampers are summarized.

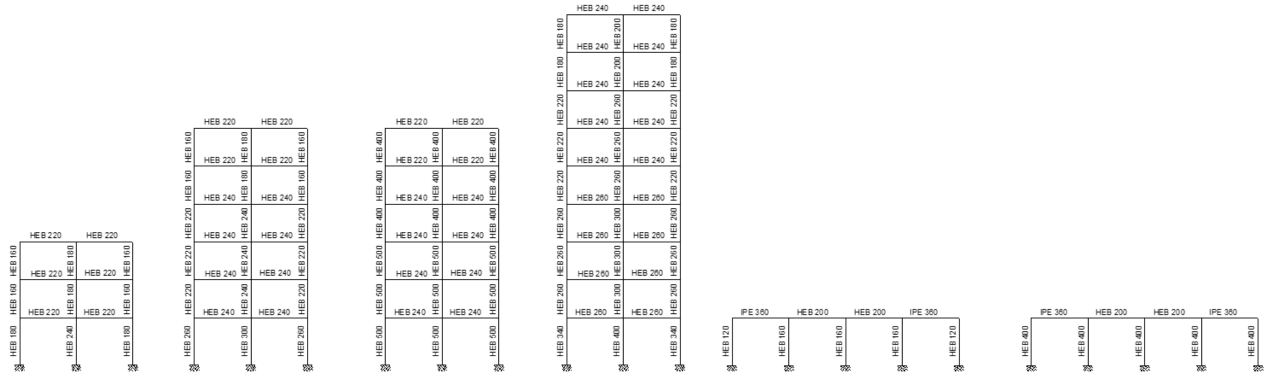

Fig. 3: The six examined moment resisting steel frames

4.1. The shear link device

The Shear Link (SL) device (fig. 4) is a double T cross section with closely spaced stiffeners, obtained from a rectangular hot laminated form which is reduced with a milling machine [7]-[11]..
The energy dissipation is obtained by the plastic deformation of the steel, mainly due to the shear stresses (Fig 5).

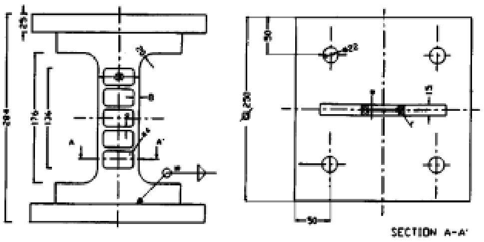

Fig. 4: Shear link device vertical and plane views [9]

During the motion, the device activates initially a shear behavior while the ultimate behavior is related to a bending response as it can be verified by observing the hysteretic curve shown in figure 5.

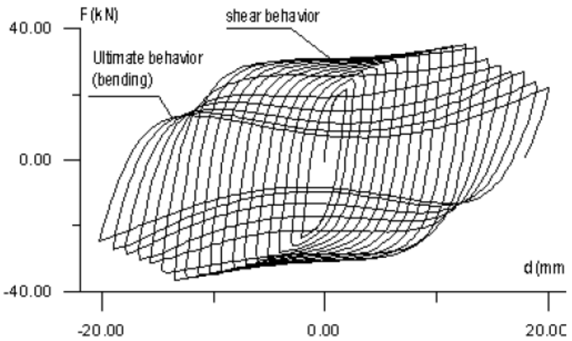

Fig. .5 Experimental hysteretic curve of the Shear link device [8]

4.2 The aluminium device

The ALuminum device (AL) is principally made of a 2 mm thick aluminum plate which is symmetrically coupled to two steel plates with wide openings that behave as lateral stiffeners of the aluminum plate (Fig. 6).

In this device, due to the low yield point of the aluminium, the panel starts to deform plastically at relatively low displacement amplitude.

In figure 7 the experimental behavior of the device subjected to a horizontal force is shown.

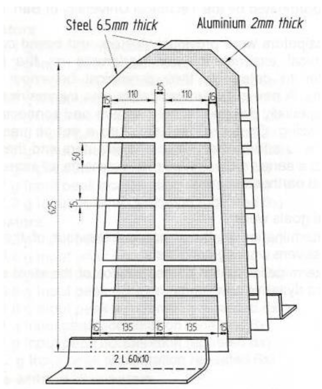

Fig. 6: Aluminum device: 3D view [13]

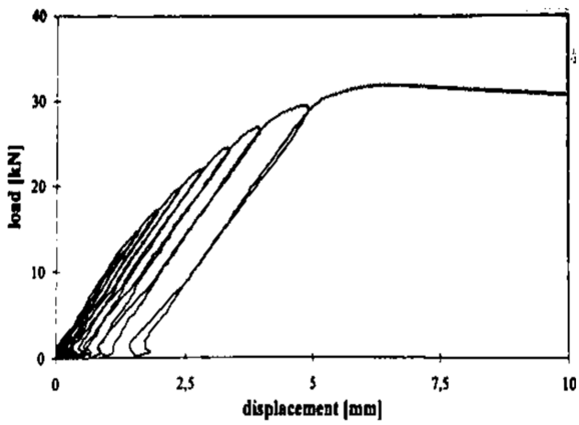

Fig.7: Experimental hysteretic curve of the Aluminum device [13]

5. The dynamic analysis

The dynamic nonlinear analysis of the aforementioned moment resisting frames subjected to the 21 near-field earthquakes (see Table 1) has been performed by means of Sap 2000 nonlinear software [14]. In detail, the passive devices have been modelled as nonlinear links whose mechanical characteristics have been defined in accordance with the experimental hysteretic behaviour shown in figures 5 and 7.
The total frame mass is due to the mass of the structural elements and to a distributed mass on each beam equal 500 kg/m.

In order to design the number of the devices, the moment resisting frames have been subjected to a distribution of horizontal forces proportional to the interstorey drifts arising from the first vibration mode of the structure, and whose resultant force is equal to 10% of the total seismic weight of the structure. The energy dissipation devices have been dimensioned assuming the yielding force F_y equal

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to a percentage of the total shear V produced at each floor by the aforementioned distribution of forces.

Four different project criteria have been considered: $F_y=20\%V$; $F_y=40\%V$; $F_y=60\%V$; $F_y=85\%V$.

The results of the dynamic analysis are summarised by means of some parameters which may be indicative of the seismic performance of the steel frames.

In detail, the global damage index (Fig. 8) has been evaluated which is defined as the ratio between the top displacement D and the total height of the structure H. The global damage index decreases as the period of the first vibration mode increases.

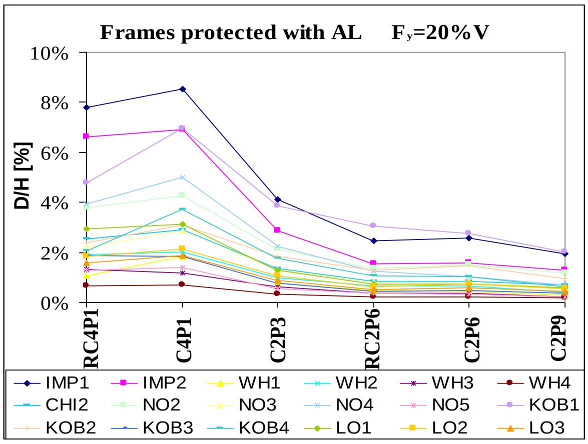


Fig. 8: Global damage index evaluated for each steel frames in fig.3 protected with the AL (designed for a yielding force equal to 20%V) and for all the near-field earthquakes in Table 1.

The described behavior has been recorded even in the cases of frames protected with SL devices (fig. 9).

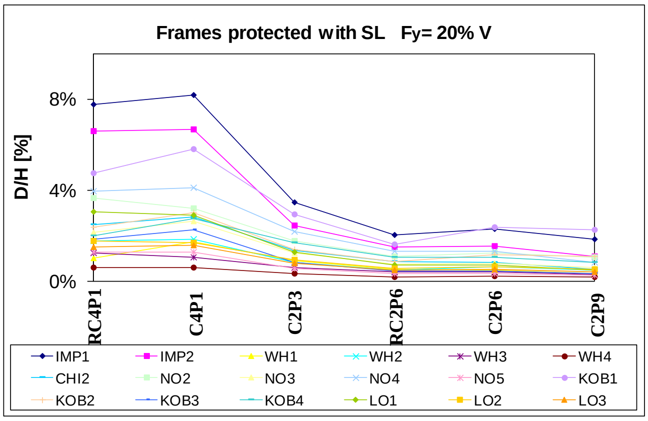


Fig. 9: Global damage index evaluated for each steel frames in fig. 3 protected with the SL (designed for a yielding force equal to 20%V) and for all the near-field earthquakes in Table 1.

The seismic analysis has been performed changing the yielding forces adopted for the design of the energy dissipation devices; the results for Imperial Valley earthquake records are shown in Fig.10. In figure 10 the "SL" indicates the presence of SL devices while the subsequent number is the yielding force adopted for the design of such device. It can be observed that the effects on the global damage index of dampers are not significant for the RC4P1 frame which is characterized by a low first fundamental period close to the pulse period of the ground motions, even for varying yielding force. For the other frames, the global damage index decreases if compared to the unprotected frames.

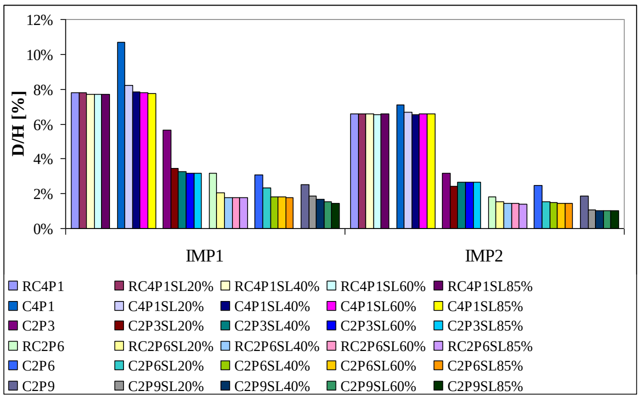
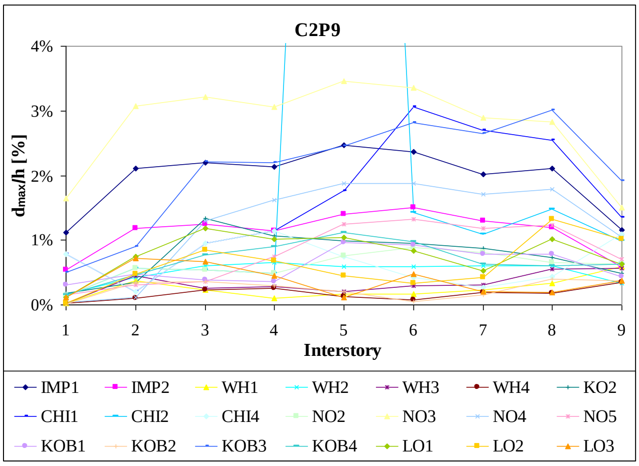


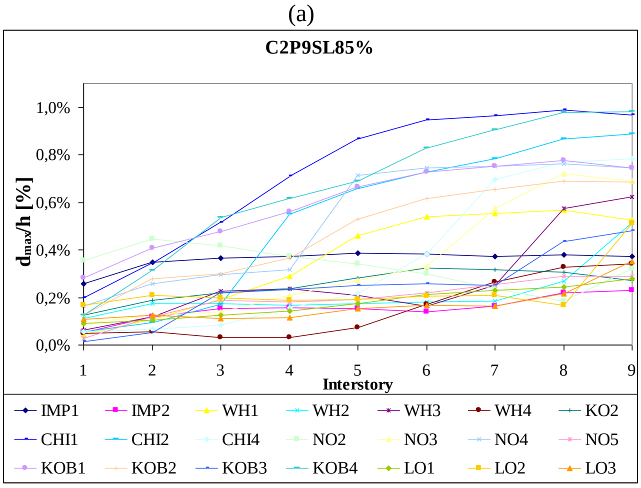
Fig. 10: Global damage index of analyzed frames with and without SL devices for Imperial Valley earthquake records.

The maximum interstorey drift index – defined as the ratio of the maximum interstorey drift d_{max} on the storey height h - has also been investigated. In Figure 11 the maximum interstorey drift index is shown for C2P9 frame, C2P9SL85% frame (corresponding to the C2P9 frame equipped with SL device dimensioned assuming the yielding force equal to 85% of shear V) and C2P9AL85% frame (corresponding to the C2P9 frame equipped with AL device dimensioned assuming the yielding force equal to 85% of shear V).

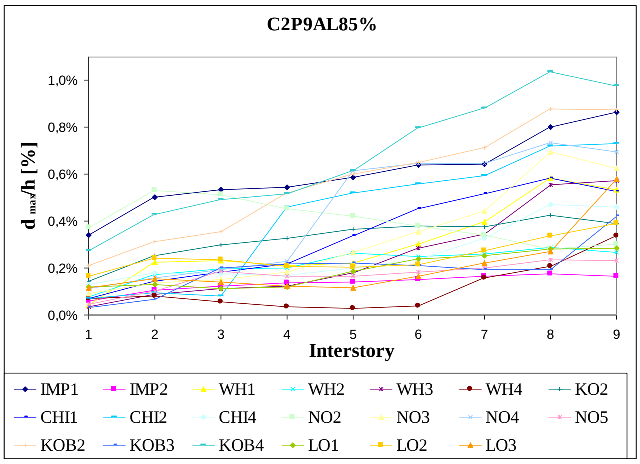
The interstorey drift index is an indicator of both structural and nonstructural damage. Figure 11 shows that both SL and AL devices are able to reduce the interstorey drifts under the value of 1,5% . In particular, the AL devices induce a better behavior at the middle height of the structure. The analysis of the other frames shows that as the height of the structure decreases, the SL devices are more efficient for limiting the interstorey drift than AL devices.



(a)



(b)



(c)

Fig. 11: Maximum interstorey drift index: a) C2P9 frame; b) C2P9 frame equipped with SL with F_y=85% V; c) C2P9 frame equipped with AL with F_y= 85%V.

Moreover, it has been observed that the frames equipped with energy dissipation devices dimensioned assuming a lower yielding force show higher interstorey drifts at lower floors and that this behavior is more evident in the case of frames protected with SL device.

In Fig. 12 the peak absolute floor accelerations are shown; this parameter is important to assess the behavior of nonstructural elements as the horizontal inertia forces at each floor are proportional to this value.

In all the examined cases, the frames equipped with AL devices exhibit a lower level of peak accelerations than the ones with SL devices. Moreover, the energy dissipated by AL device is higher than the one dissipated by SL device.

The study shows that AL devices induced a better behavior of the structure than the SL ones as the total height of the structure increases.

6. Conclusion

The present study investigates the response of moment resisting steel frames subjected to near field earthquakes. In detail, frames have been analysed by considering the installation of two different seismic devices which are characterised mainly by shear stresses and a hysteretic behaviour due to the plasticization of the central panel. As the characteristics of their hysteretic behaviour are quite different, the performances of the protected frames have been compared for 21 near field earthquakes. The dynamic analysis shows that the AL device induced a better behaviour than the SL device increasing the total height of the frame.

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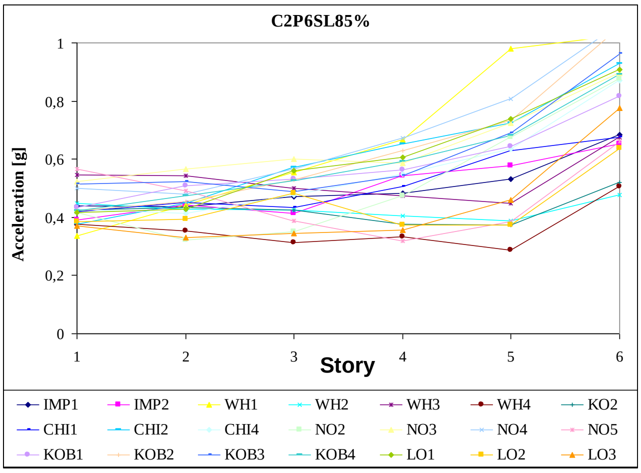
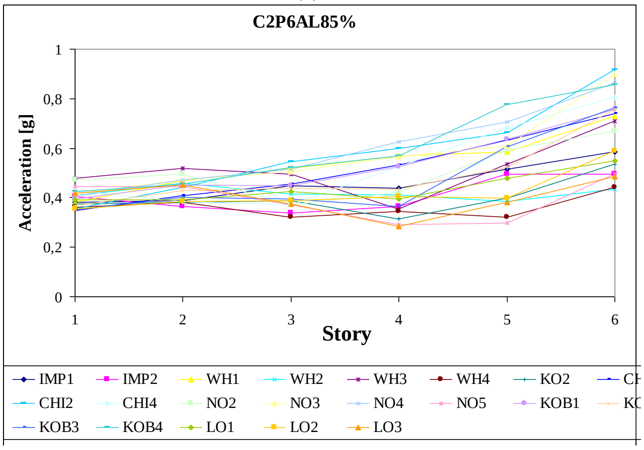


(a)



(b)

Fig. 12: Peak absolute floor accelerations for C2P6 frame a) equipped with SL with Fy=85% V; b) equipped with SL with Fy=85% V.

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