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Nabid, N., Hajirasouliha, I. and Petkovski, M. (2019) Simplified method for optimal design of friction damper slip loads by considering near-field and far-field ground motions. *Journal of Earthquake Engineering*, 25 (9). pp. 1851-1875. ISSN: 1363-2469

<https://doi.org/10.1080/13632469.2019.1605316>

This is an Accepted Manuscript of an article published by Taylor & Francis in *Journal of Earthquake Engineering* on 29/04/2019, available online:

<http://www.tandfonline.com/10.1080/13632469.2019.1605316>.

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SIMPLIFIED METHOD FOR OPTIMAL DESIGN OF FRICTION DAMPER SLIP LOADS BY CONSIDERING NEAR-FIELD AND FAR-FIELD GROUND MOTIONS

(Effect of near and far-field earthquakes on optimum design of friction dampers)

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Abstract

A simplified method is proposed for optimum design of friction dampers by considering the characteristics of design earthquakes. Optimum slip loads for 3, 5, 10, 15 and 20-storey RC frames with friction wall-dampers are obtained for a set of 20 near and far-field earthquakes as well as artificial spectrum-compatible records scaled to different acceleration levels. Optimum solutions are shown to be more sensitive to Peak Ground Velocity (PGV) than Peak Ground Acceleration (PGA), especially for near-field earthquakes with high velocity pulses. For identical PGA levels, far-field earthquakes on average result in 1.5 times lower optimum slip loads compared to near-field records, while they lead to 118% higher energy dissipation and 24% lower maximum inter-storey drifts. Empirical equations are proposed to predict optimum slip loads (as a function of number of storeys and PGA/PGV of design earthquakes) and their efficiency is demonstrated through selected examples.

Keywords: Near- and far-field earthquakes; Optimum design; Friction damper; Slip load distribution; Energy dissipation.

1. Introduction

Friction-based passive energy dissipation devices have been successfully used in practice to enhance seismic performance of both newly designed and existing structures subjected to strong earthquake excitations [Vezina and Pall, 2004; Pasquin et al., 2004; Shiraia et al. 2019]. Different types of friction-based dampers have been developed recently including friction wall dampers [Nabid et al. 2017], rotational friction dampers [Mualla and Belev, 2017], friction braced frames [Tirca et al., 2018], and posttensioned concrete walls with friction devices [Guo et al., 2017]. However, finding the optimum values of slip loads in the friction devices (the loads

33 at which the friction devices start slipping and hence dissipating energy) is challenging, since these values can
34 be sensitive to the characteristics of the seismic excitation.

35 Several research studies have been carried out on optimum design of friction dampers under earthquake
36 excitations using different optimisation techniques such as Genetic Algorithm (GA) [Moreschi and Singh, 2003;
37 Mohammadi et al., 2018], backtracking search optimisation algorithm (BSA) [Miguel et al., 2016] and Uniform
38 Distribution of Deformation (UDD) [Nabid et al., 2018], or used iterative methods to find the optimum range of
39 slip load values. However, the aforementioned optimisation approaches are computationally expensive and/or
40 require complex mathematical calculations, and therefore, may not be directly used in practical applications. On
41 the other hand, most of the existing research studies on optimum design of friction dampers have been either
42 based on a code-based design spectrum, a set of spectrum-compatible natural/synthetic earthquakes or a single
43 natural earthquake [Petkovski and Waldron, 2003; Pall and Pall, 2004; Lee et al., 2008; Shirkhani et al., 2015;
44 Nabid et al., 2017], where the effects of different types of earthquakes have been neglected. For more accurate
45 design, however, the earthquake uncertainties should be taken into account in terms of fault type, earthquake
46 intensity, peak acceleration and velocity, frequency content, duration, earthquake magnitude and distance.

47 In an early attempt, a design slip load spectrum was developed by Filiatrault and Cherry [1990] to obtain the
48 best slip load distribution for friction dampers by minimising an energy performance index while considering
49 the properties of the structure and the ground motion anticipated at the construction site. They concluded that
50 the optimum slip load is not only a structural property but also depends on the frequency and amplitude of the
51 ground motion. The values of the optimum slip loads in their study were shown to be linearly proportional to the
52 peak ground acceleration of the input earthquake. In a more recent study, Kiris and Boduroglu [2013]
53 investigated the correlation between the peak displacement demand of a RC structure with friction damper and
54 different parameters used to measure the severity of ground motions. It was demonstrated that depending on the
55 fundamental period of the frame, the strength ratio of the system at slip displacement and the soil profile,
56 different ground motion parameters can play a dominant role in the seismic response of the structure.

57 Previous studies show that structures designed using older seismic design provisions, based on far-field
58 earthquakes, may experience extensive damage or failure in case of near-field earthquakes [Alavi and
59 Krawinkler, 2001]. The main reason is that large displacement demands can be imposed to the structures by
60 severe pulses of near-field ground motions compared to the far-field earthquakes. In pulse-like ground motions,
61 the amplitude and period of the pulse in the velocity time history are the key parameters to control the
62 performance of the structures, and therefore, they should be taken into account for both design and retrofit of
63 structures in the near-field zones [Alavi and Krawinkler, 2001]. There is also displacement amplification in the
64 long-period structures caused by the large amplitudes in the long period range of displacement response spectra
65 [Anderson and Bertero, 1987]. The results of Alavi and Krawinkler [2001] study indicated that conventional
66 retrofit techniques accompanied by increasing the stiffness and/or strength of the system are not efficient for
67 long-period structures subjected to severe pulse-like earthquakes. This is due to moving the structure into a
68 range of higher spectral accelerations by increasing the stiffness (or decreasing the period) of the system. Unlike
69 the cumulative effects of far-field ground motions, the structure dissipates the earthquake input energy in few
70 large displacement excursions under near-field records, where most of the seismic input energy arrives in a

71 single long-period velocity pulse associated with forward directivity or fling step displacements and response
72 amplification of the long-period structures [Somerville, 1998].

73 Tirca et al. [2003] investigated the response of middle-rise steel moment-resisting frames with and without shear
74 link (SL) devices subjected to near-field ground motions. Based on their results, the near-field earthquakes
75 expose the structure to higher ductility demands than the far or intermediate-field ground motions. Also, they
76 showed that for the stiffer structures, the shear forces were generally higher at the upper storeys. In a study
77 performed by Xu et al. [2007], the performance of yielding and viscous passive energy dissipation systems were
78 investigated subjected to near-field ground motions by using an analytical ground velocity pulse model. They
79 concluded that the performance of different passive energy dissipation systems depends significantly on the
80 period of the pulse excitation, and therefore, to achieve the best performance, the pulse periods must be taken
81 into account when designing passive energy dissipation systems. Lin et al. [2010] evaluated the efficiency of
82 using initially accelerated passive tuned mass damper (PTMD) to reduce the dynamic responses of structures
83 under near-fault ground motion records. They showed that an appropriate PTMD initial velocity used to
84 accelerate the motion can efficiently reduce the local peak seismic responses of the system under near-fault
85 earthquakes. In another relevant study, Hatzigeorgiou and Pnevmatikos [2014] developed a straightforward
86 method for the evaluation of effective velocities and damping forces for single-degree-of-freedom (SDOF)
87 structures with supplemental viscous dampers under near-source earthquakes. Using their proposed method, it
88 was observed that the inelastic velocity ratio is strongly affected by the period of vibration, the effective viscous
89 damping ratio, the forced reduction factors and the type of seismic fault mechanism. Bhandari et al. [2017]
90 investigated the behaviour of a base-isolated building structure subjected to far-field and near-field earthquakes
91 with directivity and fling-step effects. According to their results, under the near-field earthquakes with fling-step
92 effect, the base isolation proved to be ineffective in terms of reducing base shear, top storey absolute
93 acceleration and maximum inter-storey drift. In a more recent study, Castaldo and Tubaldi (2018) investigated
94 the effects of ground motion characteristics on the optimum friction pendulum properties of seismic isolation
95 systems. It was shown that PGA/PGV is a better indicator of the frequency content of the input ground motion
96 compared to PGA, and can help to provide less scatter predictions.

97 The research on the effects of near and far-field earthquakes is mainly focussed on the efficiency of base-
98 isolated systems, viscous dampers and semi-active control devices, with few efforts in the design of friction-
99 based passive control systems subjected to the near and far-field records. This study aims to evaluate the effects
100 of near-field and far-field ground motions on optimum design of friction wall dampers leading to a maximum
101 amount of energy dissipation efficiency in friction devices. To achieve this, at first, a comprehensive parametric
102 study is performed on 3, 5, 10, 15 and 20-storey RC frames equipped with friction wall dampers (using a wide
103 range of slip load values) under spectrum compatible earthquakes scaled to different PGA levels as well as a set
104 of 20 near and far-field earthquake records. Based on the results, empirical equations are proposed to obtain the
105 optimum slip load values by considering the effects of number of storeys and earthquake PGA and PGV levels.
106 The efficiency of the proposed design method is then demonstrated through several design examples.

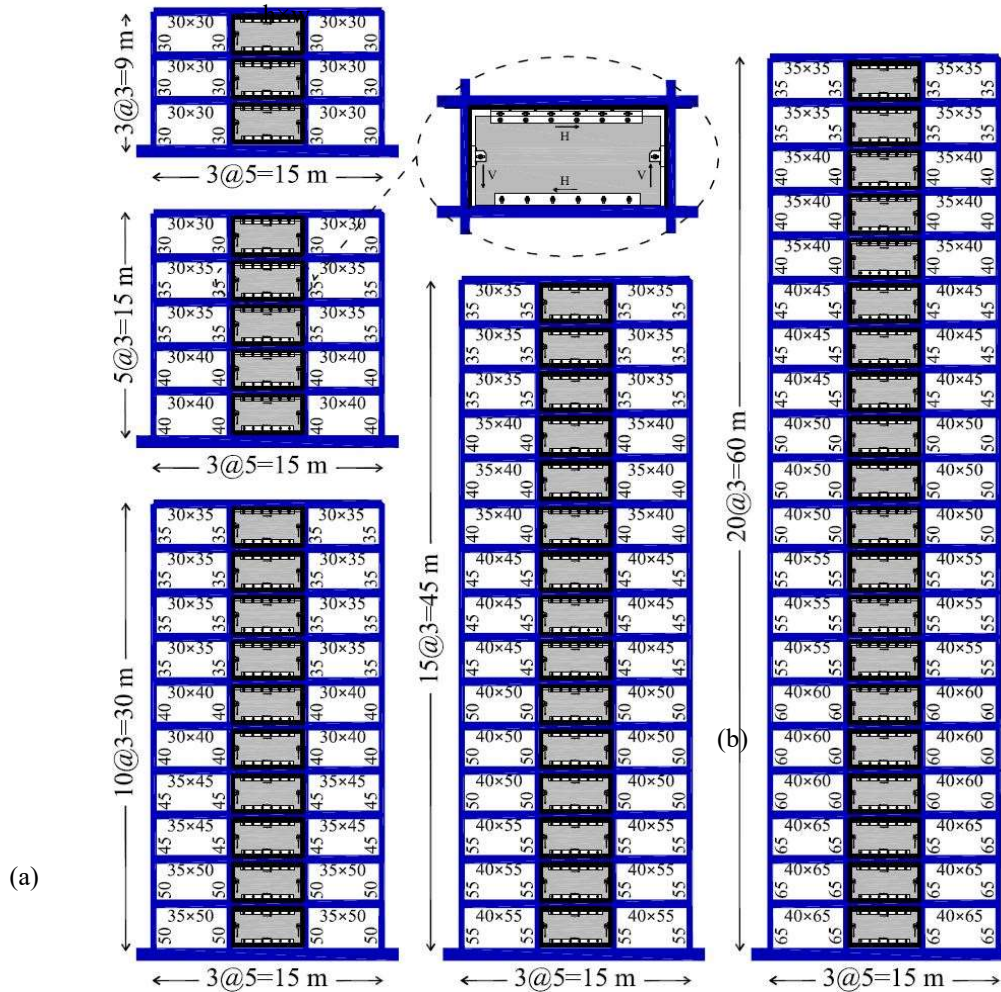
107 2. Numerical Modelling

108 To investigate the efficiency of the proposed design methodology, 3, 5, 10, 15, and 20-storey RC frames were
109 designed using the typical geometry shown in Figure 1 (a). The utilised friction damper (schematic view shown
110 in Figure 1 (b)) comprises of a reinforced concrete wall panel connected to the frame system through two
111 vertical supports in the sides, a horizontal connection at the bottom, and a friction device at the top. The
112 connections are designed to avoid transferring extra shear forces to the middle of the adjacent beam and column
113 elements. As shown in Figure 1 (b), the utilised friction device is a conventional Slotted Bolted Connection
114 (SBC) with two steel plates over a central T-shape slotted steel plate anchored to the top floor beam. It should be
115 mentioned that the concrete panels at ground level are fixed to the base to reduce the maximum axial loads in
116 the columns at the ground level. Table 1 lists the period and mass participation factor of the first three modes of
117 vibration for the frames with and without with friction wall-dampers. More detailed information about the
118 adopted friction wall damper can be found in Nabid et al. [2017].

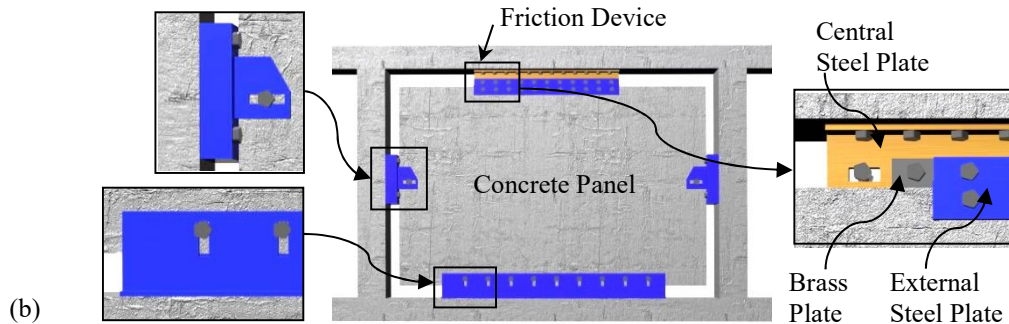
119 The designed frames were considered to be located in a low-to-medium seismicity region with PGA of 0.2 g and
120 soil type C category of Eurocode 8 [EC8; CEN, 2004a]. The uniformly distributed dead and live loads were
121 considered to be 5.5 kN/m² and 2.5 kN/m² for interior floors, and 5.3 kN/m² and 1.0 kN/m² for the roof level,
122 respectively. The frames were initially designed based on EC8 [CEN, 2004a] seismic loads and in accordance
123 with the minimum requirements of Eurocode 2 [EC2; CEN, 2004b] for moment-resisting RC frames with
124 medium ductility (DCM). The concrete compressive strength (f'_c) and the yield strength of steel reinforcement
125 bars (f_y) were assumed to be 35 MPa and 400 MPa, respectively.

126 OpenSees software [McKenna and Fenves, 2000] was used to conduct pushover and nonlinear time-history
127 analyses. Concrete sections were modelled using a uniaxial constitutive material with linear tension softening
128 (Concrete02), while the behaviour of steel bars was simulated by a Giuffre–Menegotto–Pinto model (Steel02)
129 with 1% isotropic strain hardening. Beam and column members were divided into three elements and modelled
130 using displacement-based nonlinear beam-column elements with fibre sections while four Gauss–Lobatto
131 integration points were considered for each element. The confinement effects due to the presence of transverse
132 reinforcement were taken into account in the material model of the concrete fibres using *fib* Model Code 2010.
133 P-Delta effects were taken into account in both pushover and nonlinear time-history analyses. A classical
134 Rayleigh damping model proportional to both mass and stiffness matrices (i.e. $C = \alpha M + \beta K$) was adopted and
135 a constant damping ratio of 0.05 was assigned to the first mode and to the modes at which the cumulative mass
136 participation exceeds 95%.

137



138
139



140

141 **Figure 1.** (a) Geometry of the reference RC frames equipped with friction wall dampers, (b) schematic view of
142 the friction wall damper (adopted from Nabid et al. [2017])

143 The results of the analytical studies showed that the strength of reinforced concrete wall panels with 15 cm
144 thickness is always higher than the maximum loads transferred from the friction device [Nabid, 2018].
145 Therefore, in this study the wall panels were modelled using equivalent elastic elements. An inelastic link
146 element, representing an ideal Coulomb friction hysteretic behaviour, was utilised to model the friction device.
147 The beam-to-column connections were assumed to be fully rigid with no shear failure in the panel zones. A
148 computer code in MATLAB [2014] was also developed and linked to the OpenSees [McKenna et al., 2000]

149 software to calculate the energy dissipation in the structural elements and friction devices under earthquake
 150 excitations.

151 **Table 1.** Period and mass participation factor of the first three modes of vibration

		Frames without friction dampers		Frames with friction dampers	
		Fundamental Period (sec)	Mass Participation Factor	Fundamental Period (sec)	Mass Participation Factor
3- Storey	Mode 1	0.71	82.9%	0.15	81.9 %
	Mode 2	0.22	14.1%	0.06	10.4 %
	Mode 3	0.12	2.8%	0.04	6.7 %
5- Storey	Mode 1	0.99	77.7%	0.29	73.8 %
	Mode 2	0.32	11.8%	0.08	15.7 %
	Mode 3	0.17	5.7%	0.05	5.7 %
10- Storey	Mode 1	1.56	78.1%	0.78	68.3 %
	Mode 2	0.55	10.0%	0.19	18.1 %
	Mode 3	0.31	4.3%	0.09	6.4 %
15- Storey	Mode 1	1.93	75.0%	1.29	65.1 %
	Mode 2	0.73	11.5%	0.33	17.3 %
	Mode 3	0.42	4.7%	0.15	6.1 %
20- Storey	Mode 1	2.31	73.1%	1.78	64.3 %
	Mode 2	0.84	11.1%	0.47	16.2 %
	Mode 3	0.49	4.0%	0.22	5.9 %

152

153 **3. Characteristics of Near-field Earthquakes**

154 In general, the distance of the structure from the fault rupture is one of the dominant factors influencing the
 155 imposed peak displacement demand. The near-field zones are typically considered to be within 12 km from the
 156 fault rupture, while far-field regions are those with epicentral distances of the recording stations ranging from 12
 157 to 64 km [Chopra and Chintanapakdee, 2001]. Some researchers have classified near-field zones as those within
 158 20-60 km from the fault rupture [Stewart et al., 2002]. In general, in a near-field zone and at a particular site, the
 159 earthquake characteristics are significantly influenced by three factors: the rupture mechanism, slip direction
 160 relative to the site and the residual ground displacement at the site due to the tectonic movement. Forward-
 161 directivity pulses usually occur when the rupture propagation velocity is close to the shear-wave velocity and
 162 the direction of slip on the fault is aligned with the site (mainly oriented in the fault-normal direction due to the
 163 radiation pattern of the fault) [Somerville and Smith, 1996; Bray and Rodriguez-Marek, 2004; Davoodi and
 164 Sadjadi, 2015]. Due to forward-directivity effect, large-amplitude pulses of motion are generated with long
 165 period (1–1.5 s) and short duration while having a high ratio of Peak Ground Velocity (PGV) to Peak Ground
 166 Acceleration (PGA) [Somerville et al., 1997]. Therefore, in near-field areas high velocity pulses, which are
 167 extremely destructive in nature, are one of the main factors to define the severity of the seismic input rather than
 168 the PGA value. Regarding the last factor, the tectonic deformation associated with the fault rupture may contain
 169 a significant permanent static displacement termed fling-step effect [Bray and Rodriguez-Marek, 2004]. It
 170 produces a high amplitude velocity pulse and a monotonic step in the displacement time history [Somerville,
 171 2002]. Additionally, hanging wall and footwall effects can be observed in dipping fault earthquakes. The fault

172 plane has generally closer proximity to the sites on the hanging wall than the sites on the footwall at the same
173 distance. The hanging wall sites have larger amplitude and slower attenuation in ground motion parameters than
174 the footwall sites with the same distance. These effects have higher influence on the acceleration spectra in short
175 periods. The aforementioned fling-step effect is the relative slip between the hanging wall and footwall
176 [Abrahamson and Somerville, 1996].

177 Near-field earthquakes transfer a major portion of fault energy in the form of pulses, which can be frequently
178 seen in displacement, velocity, and acceleration time histories. These pulses tend to have high Fourier spectrum
179 in limited periods, while in far-field earthquakes the high Fourier spectrum generally occurs in broad range of
180 periods [Iwan, 1994; Bhandari et al., 2017]. In the frequency domain, depending on the fault-normal or fault-
181 parallel components of the forward-directivity ground motions in near-field region, near-field earthquakes can
182 have either higher or lower frequency contents compared to the far-field earthquakes [Davoodi and Sadjadi,
183 2015]. Davoodi and Sadjadi [2015] also showed that the maximum Fourier amplitudes of far-field earthquakes
184 and fault-parallel component of forward-directivity ground motions are distributed at higher frequencies (mostly
185 beyond 1Hz) compared to the maximum Fourier amplitudes of near-field earthquakes with fling-step and fault-
186 normal component of forward-directivity records which generally occurs at frequencies less than 1Hz. It was
187 demonstrated by Malhotra [1999] that, for the same PGA and duration of shaking, ground motions containing
188 directivity pulses can result in much higher base shear, inter-storey drift, and roof displacement in high-rise
189 structures as compared to those without pulses.

190 **4. Ground Motion Datasets**

191 *4.1. Natural Near-field and Far-field Earthquake Records*

192 In this study, two sets of 10 near-field and 10 far-field ground motions were used to evaluate the seismic
193 performance of the 3, 5, 10, 15 and 20-storey frame structures with friction wall dampers. All the selected
194 ground motions correspond to soil class C of EC8 with surface magnitudes ranging from 6.5 to 7.4. Tables 2 and
195 3 list the designations and characteristics of the selected unscaled near-field and far-field ground motions,
196 respectively. The rupture distances (R: distance from the fault rupture plane to the site) are within 10 km for the
197 near-field records and between 12 and 30 km for the selected far-field ground motions. The fault rupture
198 mechanisms are strike slip and reverse for all the records. It should be noted that the forward directivity effect of
199 the near-field ground motions generally leads to more intense fault-normal component compared to the fault-
200 parallel component [Somerville, 1998]. In this study, the fault-normal components with higher intensities were
201 selected for the nonlinear time history analyses.

202 Figures 2 (a) and (b) compare the 5% damped elastic acceleration and velocity response spectra of the studied
203 unscaled near-field and far-field earthquakes, respectively. The acceleration response spectra show that the
204 mean spectrum of the near-field earthquakes is well above whereas the far-field mean spectrum is well below
205 the EC8 design spectrum. This implies that, with the same range of surface magnitudes, the intensity of near-
206 field records is much higher than those recorded far away from the earthquake epicentre. Although the elastic
207 acceleration response spectrum provides the basis to identify the characteristics of the design earthquakes, in
208 case of near-field ground motions, the acceleration response spectrum does not adequately characterise the

209 design earthquake. This is because near-field earthquakes are mainly characterised by a relatively long period
 210 pulse of strong motion with fairly short duration, while the far-field motions have relatively long duration
 211 [Somerville, 1998]. Therefore, to better show the characteristics of the selected earthquakes, the elastic velocity
 212 response spectra of the selected near-field and far-field earthquakes with their mean spectra are also shown in
 213 Figure 2.

214 **Table 2.** Properties of the selected near-field ground motions

No.	Earthquake	Ms	Station	Abr.	R (km)	PGA (g)	PGV (cm/s)	PGV/PGA (s)
1	1999 Duzce	7.14	Duzce	DUZ	6.58	0.515	84	0.166
2	1992 Erzincan	6.69	Erzincan	ERZ	4.38	0.387	107	0.282
3	1994 Northridge	6.69	Rinaldi Receiving Sta	RIN	6.50	0.874	148	0.173
4	1994 Northridge	6.69	Newhall - Fire Sta	NEW	5.92	0.590	97	0.168
5	1994 Northridge	6.69	LA - Sepulveda VA Hospital	SEP	8.44	0.932	76	0.083
6	1995 Kobe	6.9	KJMA	JMA	0.96	0.630	76	0.123
7	1995 Kobe	6.9	Takatori	TAK	1.47	0.671	123	0.187
8	1995 Kobe	6.9	Port Island	POR	3.31	0.290	51	0.179
9	1979 Imperial Valley	6.53	Meloland Geot. Array	MEL	0.07	0.298	93	0.168
10	1979 Imperial Valley	6.53	El Centro Array #4	ARR4	7.05	0.484	40	0.084

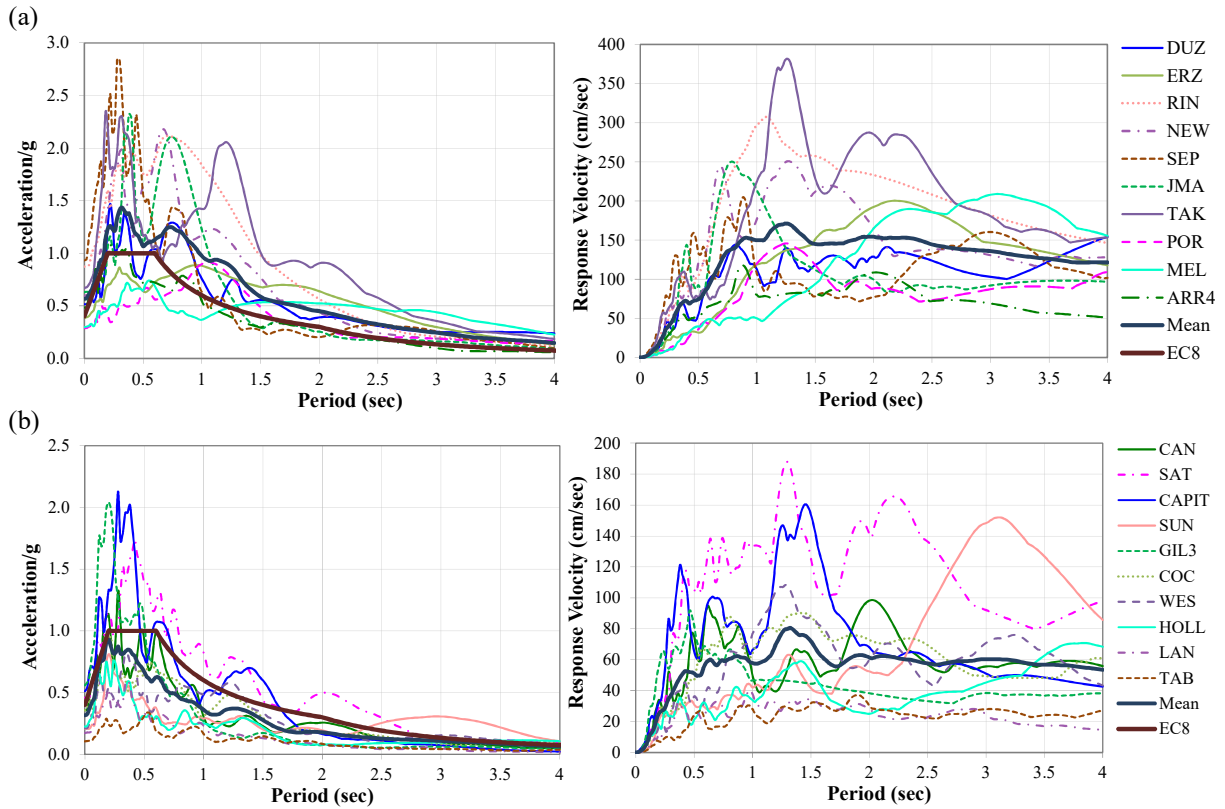
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216 **Table 3.** Properties of the selected far-field ground motions

No.	Earthquake	Ms	Station	Abr.	R (km)	PGA (g)	PGV (cm/s)	PGV/PGA (s)
1	1994 Northridge	6.69	Canoga Park-Topanga Can	CAN	14.70	0.358	34	0.097
2	1994 Northridge	6.69	Northridge-Saticoy St	SAT	12.09	0.459	60	0.133
3	1994 Northridge	6.93	Capitola	CAPIT	15.23	0.511	38	0.076
4	1989 Loma Prieta	6.93	Sunnyvale-Colton Ave.	SUN	24.23	0.207	37	0.182
5	1989 Loma Prieta	6.93	Gilroy Array #3	GIL3	12.82	0.559	36	0.066
6	1987 Superstition Hills	6.54	El Centro Imp. Co. Cent	COC	18.20	0.357	48	0.137
7	1987 Superstition Hills	6.54	Westmorland Fire Station	WES	13.03	0.211	32	0.155
8	1971 San Fernando	6.61	LA - Hollywood Stor Lot	HOLL	22.77	0.225	22	0.100
9	1992 Landers	7.28	Desert Hot Springs	LAN	21.78	0.171	19	0.113
10	1978 Tabas	7.35	Boshrooyeh	TAB	28.79	0.106	13	0.125

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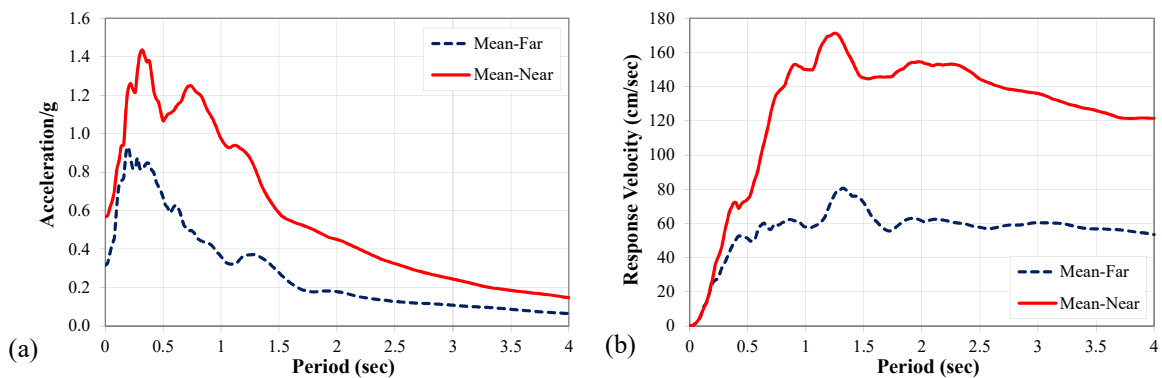


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221 **Figure 2.** Elastic acceleration and velocity response spectra of the selected (a) near-field and (b) far-field
 222 earthquakes and the EC8 design spectrum, 5% damping ratio

223 Figures 2(a) and (b) show that while the near-field ground motions have a narrower velocity-sensitive region at
 224 longer periods, they have wider acceleration-sensitive region compared to the far-field excitation records
 225 (except SUN). These results are in agreement with those obtained from the research carried out by Chopra and
 226 Chintanapakdee [2001] and Hall et al. [1995]. Figure 3 compares the mean acceleration and velocity response
 227 spectra of the selected near and far-field ground motions showing significantly higher values for the near-field
 228 records.



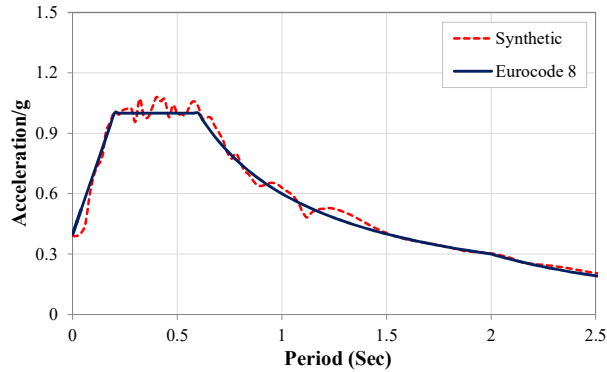
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230 **Figure 3.** Mean (a) acceleration and (b) velocity response spectra of the selected near-field and far-field
 231 earthquakes, 5% damping ratio

232 4.2. Synthetic Earthquake Record

233 The previous research by Nabid et al. [2017, 2018] implied that the earthquake uncertainty, in terms of
 234 acceleration response spectra, can be efficiently managed by using synthetic earthquakes representing the

235 average spectrum of a selected set of natural earthquakes. Therefore, a synthetic earthquake is generated using
236 the TARSCETHS [Papageorgiou et al., 2002] program to be compatible with EC8 design response spectrum for
237 high seismicity regions (i.e. PGA=0.4g) and soil class C. Figure 4 shows the good agreement between the elastic
238 acceleration response spectrum of the simulated earthquake record and the corresponding EC8 design spectrum.



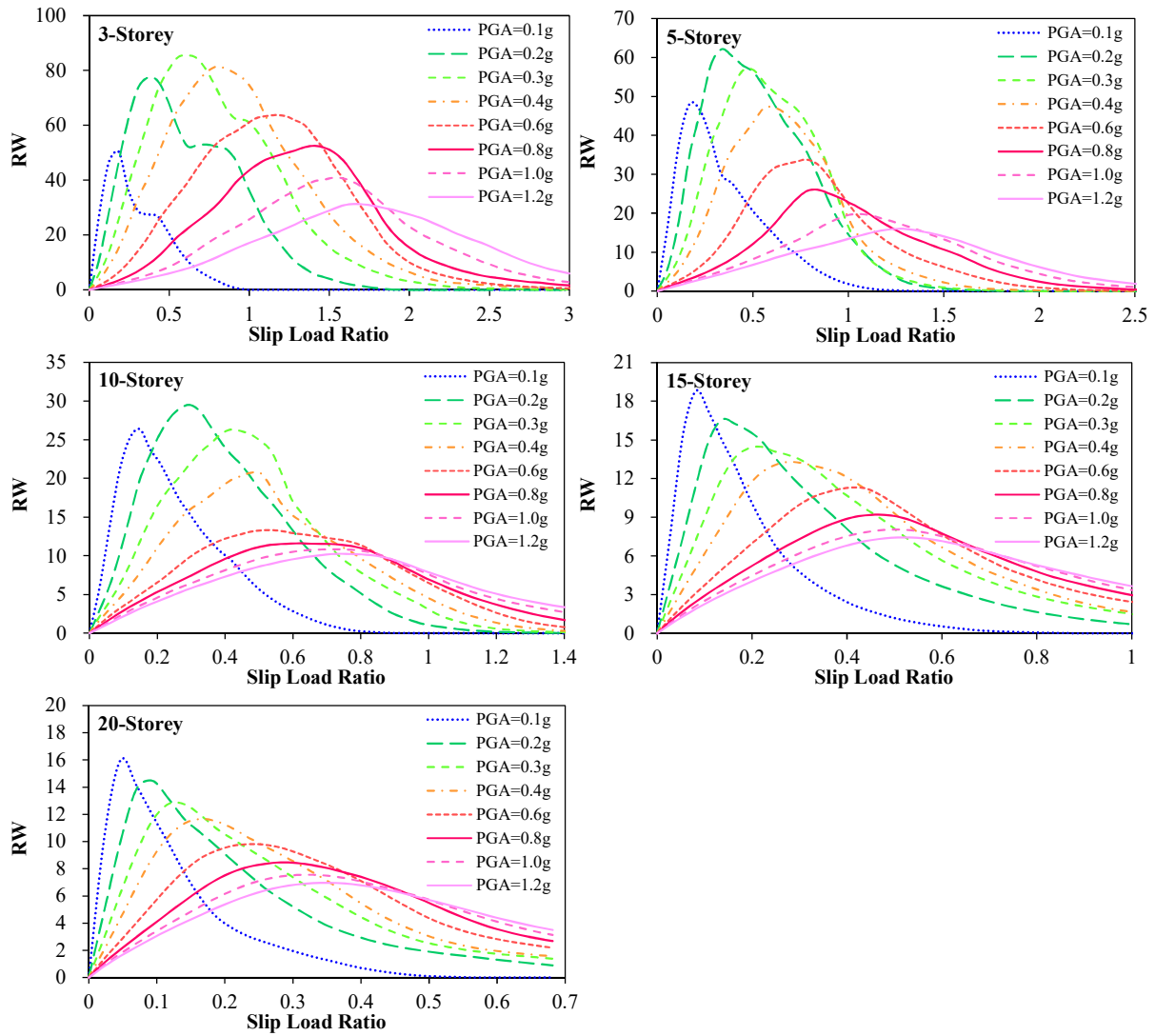
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240 **Figure 4.** Elastic acceleration response spectra of the synthetic earthquake record and the EC8 design spectrum,
241 5% damping ratio

242 **5. Effect of Earthquake Intensity Level on Energy Dissipation Efficiency**

243 The synthetic earthquake (compatible with the EC8 spectrum) was utilised to investigate the effect of the peak
244 ground acceleration (PGA) on the maximum energy dissipation efficiency of the selected frames with friction
245 wall dampers. It should be noted that different ground motion parameters may play role in peak structural
246 response demand depending on the system properties and the soil profile (Kiris and Boduroglu, 2013).
247 However, PGA is one of the widely accepted intensity measure parameters that can generally show the pattern
248 of the observed intensities (Wald et al., 1999). The energy dissipation parameter, RW, which is the ratio
249 between the work of the friction devices to the work of the beam and column elements (introduced in [Nabid et
250 al., 2017]), is considered as an effective factor for assessing the efficiency of the proposed friction wall dampers.
251 Figure 5 shows the relationships between the slip load ratio (ratio between the average of slip loads and the
252 average of storey shear strengths at all storey levels) and the energy dissipation parameter, RW, for the 3, 5, 10,
253 15 and 20-storey frames, subjected to the selected synthetic earthquake with a range of different PGA values
254 (ranging from 0.1g to 1.2g). The optimum value of the slip load ratio is considered to be the one at which the
255 RW factor reaches its peak. The results in Figure 5 show that for stronger earthquakes (higher PGA levels) the
256 optimum slip load ratios are higher and distributed over a wider range. It is also shown that the energy
257 dissipation efficiency (RW) is generally increased for lower earthquake intensity levels. This can be attributed to
258 the fact that most structural elements remain in the elastic (or near elastic) range under low intensity
259 earthquakes. The results also show a clear difference between the optimum ranges of slip load values for
260 structures with different number of storeys as will be taken into consideration in the empirical equations
261 proposed in the following sections.

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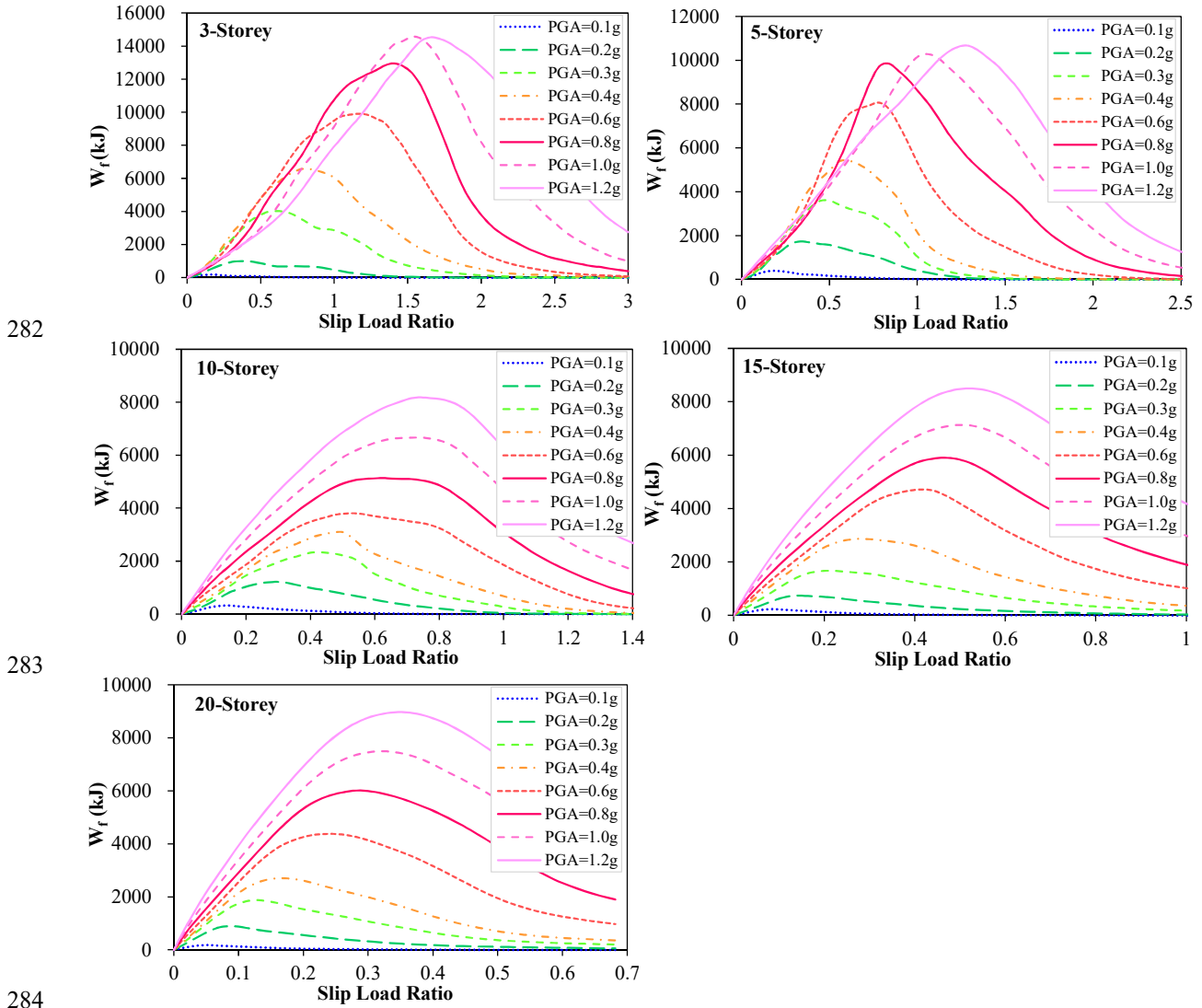
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266 **Figure 5.** Variation of energy dissipation parameter, RW, of the 3, 5, 10, 15 and 20-storey frames as function of
 267 slip load ratio under a synthetic earthquake record with different PGA levels

268 The results of a previous study [Pall and Pall , 2004] suggested that variations up to $\pm 20\%$ of the optimum slip
 269 load do not significantly affect the response; however, the range of these variations depend on the earthquake
 270 intensity. The results in Figure 5 imply that this is true for high PGA levels but not for low PGA levels, where
 271 the optimum response is significantly affected by small variations in the optimum slip load ratio. The energy
 272 dissipation effectiveness of the friction wall dampers in low to medium-rise structures initially increases with
 273 the increase of earthquake intensity up to a certain level. For the high-rise structures (15 and 20-storey);
 274 however, RW decreases monotonically by increasing the PGA. This can be mainly caused by the high stiffness
 275 of the low-rise building that in turn leads to smaller deformation demands under low PGA level earthquakes,
 276 and therefore, less energy dissipation through the work of the friction devices. This is more highlighted for the
 277 3-storey frames with almost 70% higher RW for the 0.3g input compared with that for 0.1g. Figure 6 illustrates
 278 the variation of the energy dissipated through the work of the friction devices in the 3, 5, 10, 15 and 20-storey
 279 frames under the selected synthetic spectrum compatible earthquake with different PGAs. As can be observed, a

280 negligible amount of energy is dissipated by the friction dampers in the 3 and 5-storey frames under the 0.1g
 281 earthquake compared to the other earthquake PGA levels.

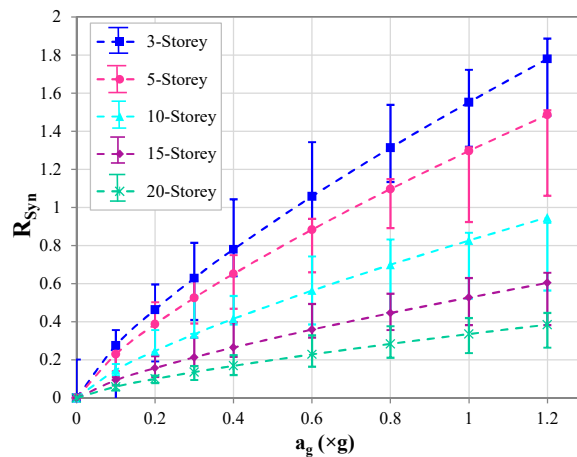


285 **Figure 6.** Variation of work of the friction devices for the 3, 5, 10, 15 and 20-storey frames as function of slip
 286 load ratio under a synthetic earthquake record with different PGA levels

287 In this study, the slip load ratios for which the energy dissipation parameter, RW, is greater than 90% of the its
 288 maximum value (i.e. less than 10% reduction), are considered as the optimum practical design range. The
 289 median (middle point) of the optimum slip load ratio ranges for the selected frames is then calculated under the
 290 synthetic spectrum-compatible earthquake using different PGA levels. Based on regression analysis using the
 291 median points, the following equation is suggested to calculate the optimum slip load ratio as a function of the
 292 earthquake PGA and number of storeys:

$$R_{Syn} = (1.16 \times e^{-0.09n} \times (a_g)^{0.75}) / 100 \quad (1)$$

293 where R_{Syn} is the optimum slip load ratio obtained for the selected spectrum-compatible synthetic earthquake
 294 (see Figure 4) and defined as the ratio between the average of slip loads and the average of storey shear
 295 strengths at all storey levels; n is the number of storeys and a_g is the PGA of the design earthquake in cm/s^2 . It
 296 should be mentioned that, to avoid using very small constant coefficients, the proposed equation is divided by a
 297 100 term. The proposed equation, on average, leads to relatively small errors (9.8%) compared to the results
 298 obtained from the parametric study on 3, 5, 10, 15 and 20-storey frames with friction wall dampers. Figure 7
 299 shows the slip load design curves obtained from equation 1 and the corresponding optimum slip load ranges as a
 300 function of earthquake intensity (PGA). While Filiatrault and Cherry [1990] suggested that the value of the
 301 optimum slip load is linearly proportional to the PGA level, the results of this study show a non-linear
 302 relationship between the PGA and optimum slip load values.

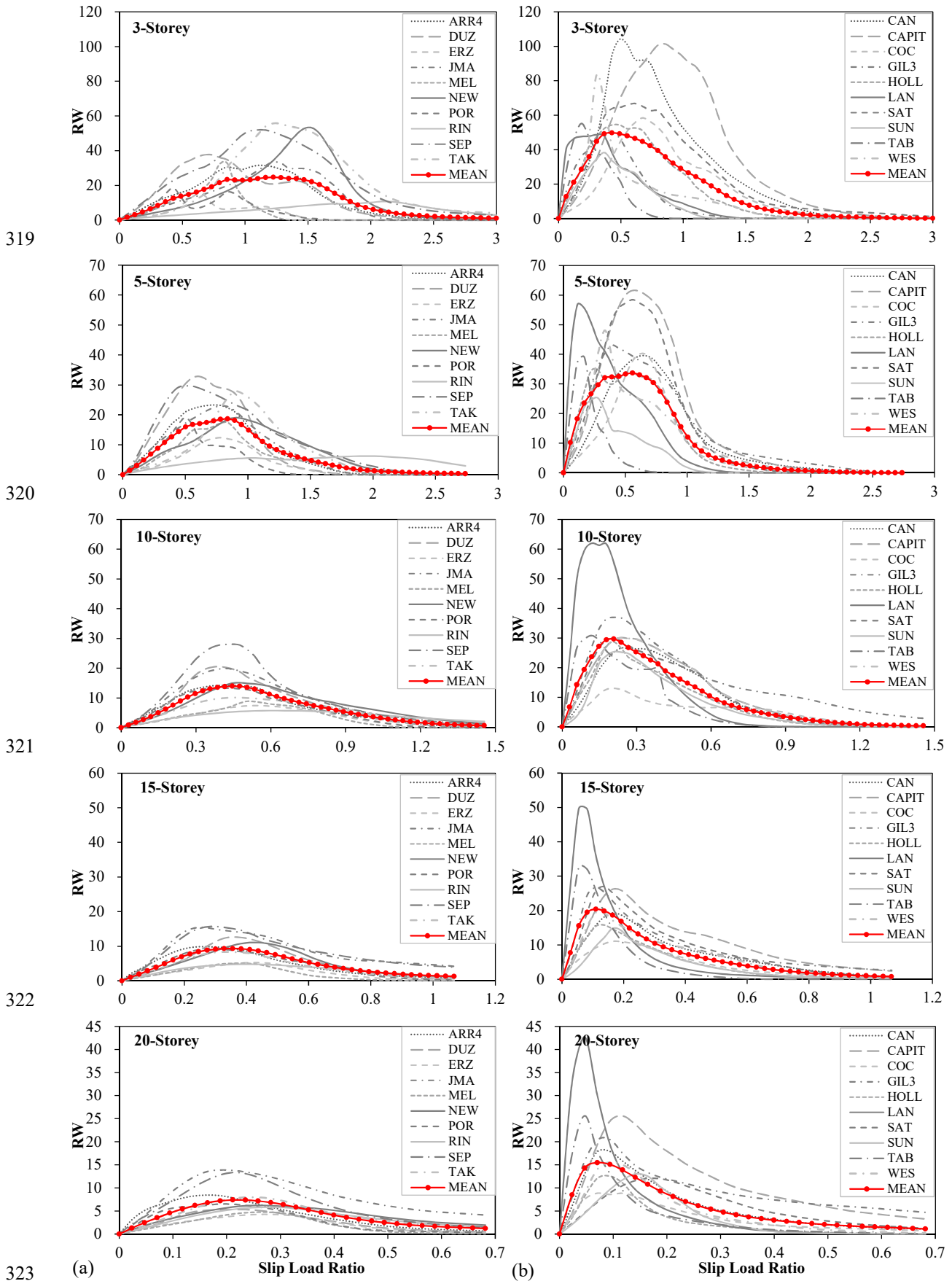


303
 304 **Figure 7.** Design slip load ratios for 3, 5, 10, 15 and 20-storey frames as a function of earthquake PGA
 305

306 While PGA is the parameter most commonly used to identify earthquake intensity, it has also been reported that
 307 it is not a totally reliable parameter to assess the seismic performance of structures. For instance, according to
 308 Housner and Jennings [1982], peak ground velocity (PGV) can be a better parameter due to its direct connection
 309 to energy demand. On the other hand, near-fault impulsive ground motions are often characterised by PGV
 310 [Malhotra, 1999; Bray and Rodriguez-Marek, 2004]. For this reason, in the following sections the effect of near-
 311 field and far-field earthquake ground motions with variable ranges of both PGA and PGV is investigated in the
 312 optimum design of friction dampers.

313 **6. Effect of Near-Field and Far-Field Earthquakes on Optimum Design of** 314 **Friction Dampers**

315 To evaluate the effect of near-field and far-field ground motions on optimum design solutions, the 3, 5, 10, 15
 316 and 20-storey frames with friction wall dampers are subjected to the natural records listed in Tables 2 and 3.
 317 Figures 8 (a) and 8 (b) present the energy dissipation parameter RW , as a function of slip load ratio for the
 318 frames under the selected near-field and far-field earthquakes, respectively.



324 **Figure 8.** Variation of energy dissipation parameter, RW, of the 3, 5, 10, 15 and 20-storey frames as function of
 325 slip load ratio under (a) near-field and (b) far-field ground motions

326 The comparison of peaks of the mean-value curves shows about twice higher energy dissipation efficiency of
327 the friction dampers for far-field earthquakes (i.e. 2.02, 1.81, 2.13, 2.18 and 2.09 for the 3, 5, 10, 15 and 20-
328 storey frames, respectively). By considering the acceleration and velocity response spectra of the earthquakes
329 (see Figure 2), those records with more intense velocity pulse and/or higher response acceleration (such as SEP,
330 TAK, RIN and JMA from the near-field set, and CAPIT, SAT and GIL3 from the far-field set of earthquakes),
331 in general, result in maximum energy dissipation efficiency at higher slip load ratios. The earthquakes with
332 relatively high velocities at longer periods (e.g. MEL and SUN) led to higher optimum ranges of slip load ratios
333 for taller buildings, compared to their corresponding mean curves; whereas for the low to medium-rise
334 structures their optimum ranges are close to those of the mean curves. On the contrary, for the low to medium-
335 rise frames, the earthquakes with the maximum velocity at lower periods (e.g. RIN) resulted in very high
336 optimum slip load ratios. This is due to the earthquake high velocity occurring at the periods close to the natural
337 period of the structure, and therefore, due to dynamic magnification effects, higher friction forces are required
338 for optimum performance of the structure.

339 By considering no more than 10% reduction in the maximum of the mean RW curves, the range of optimum slip
340 load ratios for the selected near-field earthquakes can be defined as 0.89-1.51, 0.56-0.95, 0.34-0.54, 0.25-0.44
341 and 0.17-0.28 for the 3, 5, 10, 15 and 20-storey frames, respectively. The corresponding optimum slip load ratio
342 ranges obtained for the far-field earthquakes are 0.31-0.67, 0.33-0.73, 0.16-0.27, 0.09-0.16 and 0.06-0.12. The
343 results indicate that the near-field earthquakes with higher velocity levels generally lead to higher and wider
344 optimum ranges of slip load ratios for the supplemental friction-based energy dissipation devices compared to
345 the far-field ground motions.

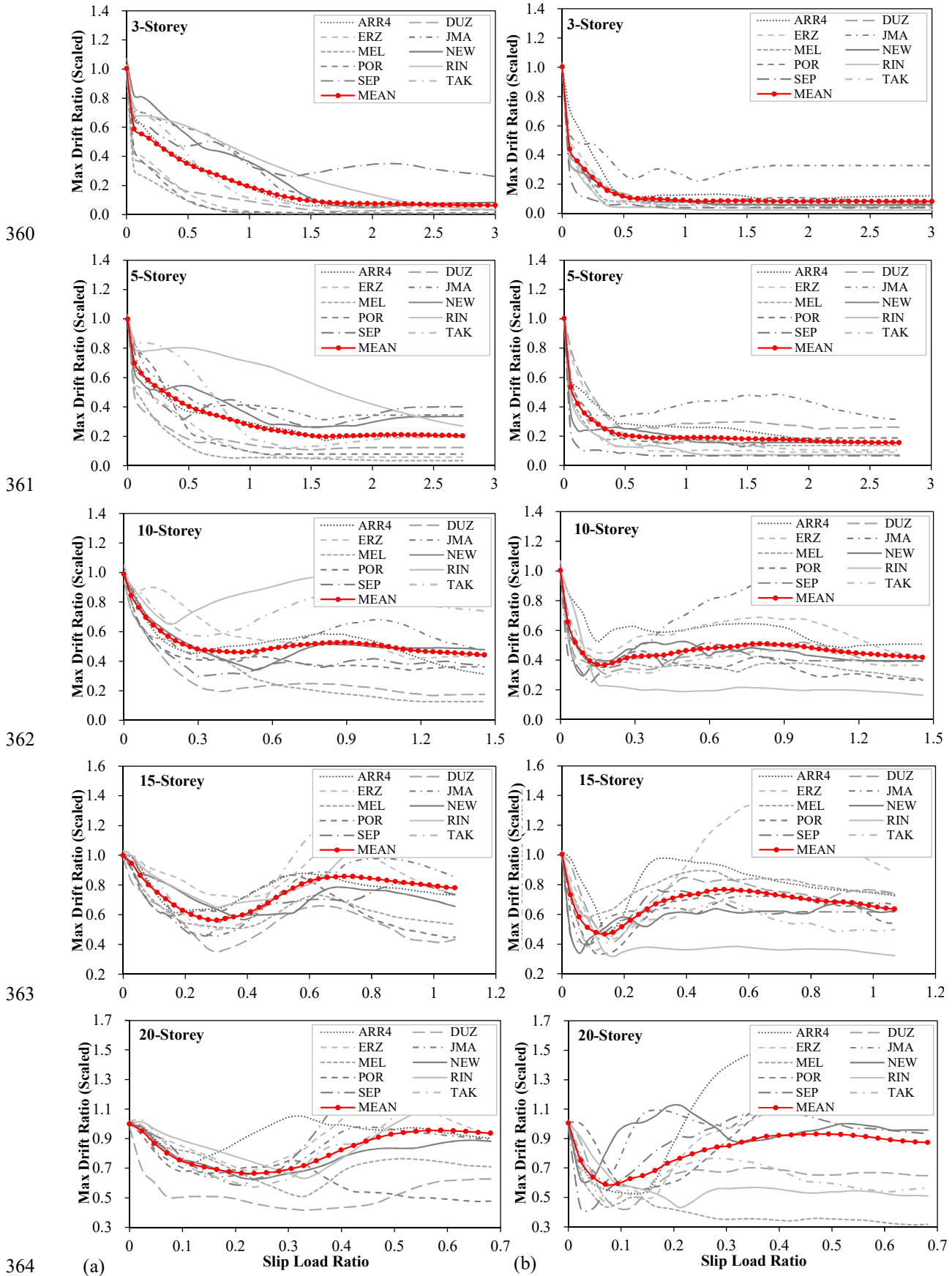
346 Figure 9 shows the variation of maximum inter-storey drift ratio of the 3, 5, 10, 15 and 20-storey frames as
347 function of slip load ratio under the selected near-field and far-field earthquakes. It should be noted that, for
348 better comparison, the results in Figure 9 are scaled to the maximum inter-storey drifts of the corresponding
349 bare frames. It is shown that the optimum slip load ratio ranges, defined earlier as those leading to the maximum
350 energy dissipation efficiency, also result in minimum drift ratios. The maximum inter-storey drift ratios were, on
351 average, attenuated by 94%, 80%, 55%, 44% and 34% for the 3, 5, 10, 15 and 20-storey frames under the near-
352 field earthquakes, respectively; and by 92%, 85%, 63%, 54% and 42% for the 3, 5, 10, 15 and 20-storey frames
353 under the far-field earthquakes, respectively. In general, the reduction in drift ratios was more noticeable in far-
354 field earthquakes, with the difference between near and far-field increasing with the increase in height of the
355 buildings (i.e. by maximum 24% reduction in 20-storey frame).

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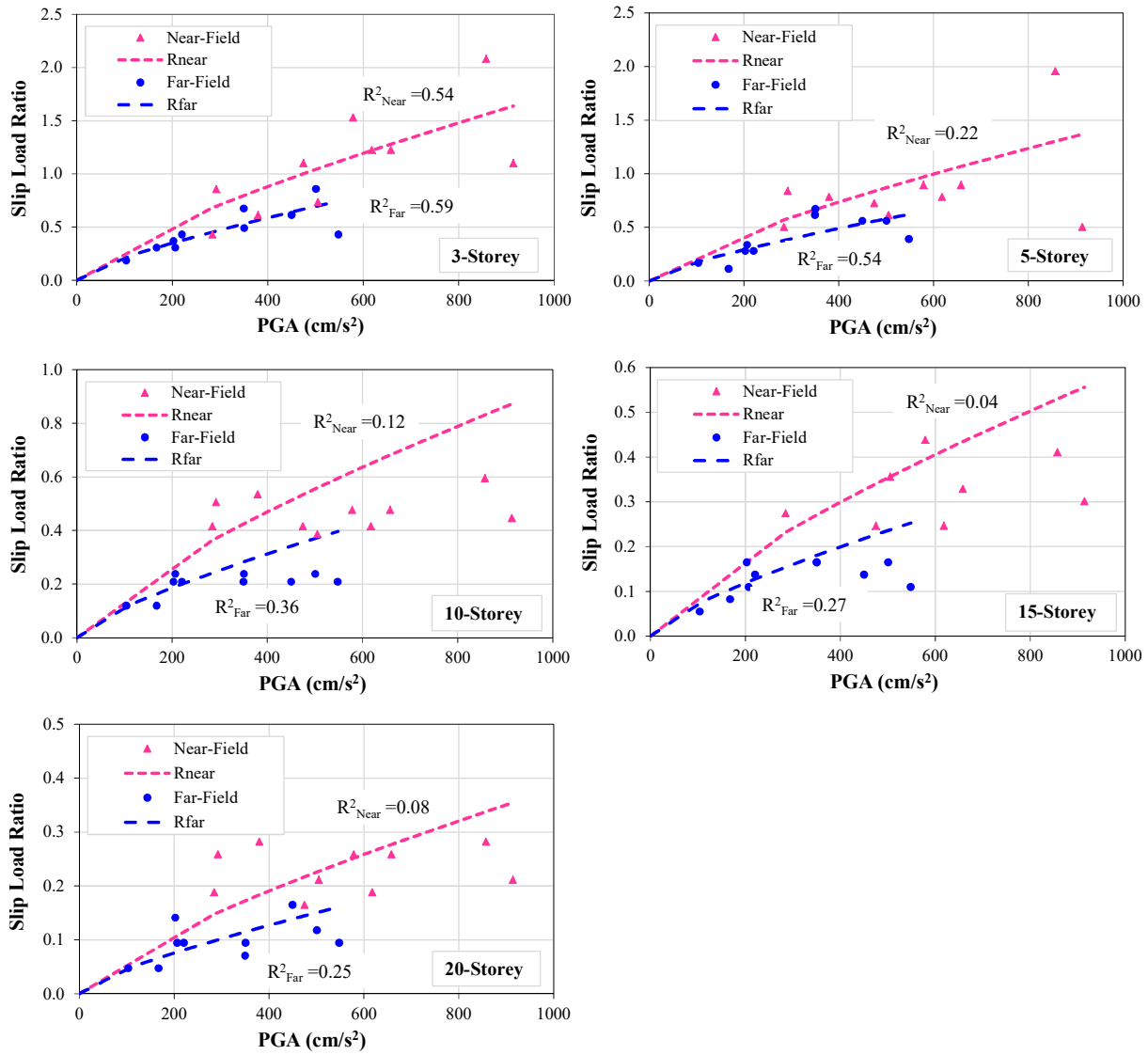
365 **Figure 9.** Variation of maximum inter-storey drift ratio (scaled to the bare frame) of the 3, 5, 10, 15 and 20-
 366 storey frames as function of slip load ratio under (a) near-field and (b) far-field ground motions

367 Using individual optimum slip load ratios corresponding to the maximum energy dissipation efficiency obtained
 368 for each near-field and far-field earthquake with specific PGA, Equation 1 can be modified to the following
 369 equations:

$$R_{near} = (1.29 \times e^{-0.09n} \times (a_g)^{0.75}) / 100 \quad (2)$$

$$R_{far} = (0.86 \times e^{-0.09n} \times (a_g)^{0.75}) / 100 \quad (3)$$

370 where R_{near} and R_{far} are the optimum slip load ratios estimated for near-field and far-field earthquakes,
 371 respectively. Figure 10 shows the variation of optimum slip load ratios of the 3, 5, 10, 15 and 20-storey frames
 372 as function of earthquake PGA for the near and far-field earthquakes overlaid with their corresponding design
 373 curves (Equations 2 and 3). Equations 2 and 3 are proposed to have, on average, minimum errors (i.e. 27% and
 374 23%) to the optimum results obtained for the near and far-field earthquakes, respectively.



375

376

377

378 **Figure 10.** Variation of optimum slip load ratio of the 3, 5, 10, 15 and 20-storey frames as function of
 379 earthquake PGA level for the near-field and far-field earthquakes with their corresponding design
 380 curves (Eq. 2 and 3)

381 For better comparison, the R-squared values are also calculated for Equations 2 and 3 using the results of the
 382 near-field and far-field earthquake records as shown in Fig. 10. It can be seen that in general the proposed
 383 equations could not accurately explain the variability of the slip load ratio data as a function of PGA. Especially
 384 there are high dispersions of the results (i.e. very low R-squared values) around the proposed equations for the
 385 high-rise frames under near-field earthquakes.

386 Based on Equations 2 and 3, for the same earthquake PGA, on average, near-field earthquakes result in 1.5
 387 times higher optimum slip loads than those for far-field earthquakes. The reason for this is the higher PGV
 388 levels of the near-field earthquakes compared to the far-field records. For example, DUZ from the near-field set
 389 of earthquake versus GIL3 from the far-field (Table 2) have PGAs of 0.515g and 0.559g, and PGV of 84m/s and
 390 36m/s, respectively. As outlined in Zhu et al. [1988] and Pavel and Lungu [2013], the PGV/PGA ratio can be
 391 used as an indicator of both frequency content of strong ground motions and potential structural damage. They
 392 revealed that low PGV/PGA ratios generally correspond to ground motions with a high frequency content in the
 393 strong-motion phase (e.g. SEP, CAPIT and GIL3), whereas high PGV/PGA ratios, in general, are associated
 394 with the ground motions with intense, long-duration acceleration pulses (e.g. TAK and NEW). In pulse-like
 395 ground motions, the coherent long-period pulses may lead to the PGV/PGA ratio of ground motions become
 396 larger (e.g. ERZ and TAK). Therefore, the ground motions with higher PGV/PGA values generally have larger
 397 damage potential [Meskouris et al., 1992]. Ground motions at moderate distances from the energy source
 398 normally have a broad range of significant frequency content, resulting in intermediate PGV/PGA ratios (e.g.
 399 TAB and LAN).

400 Figure 11 shows the optimum slip load ratios of the 3, 5, 10, 15 and 20-storey frames under the selected near
 401 and far-field earthquakes as function of the earthquake PGV/PGA ratios. For similar PGV/PGA ratios, the
 402 earthquakes with higher values of PGA and PGV result in higher optimum slip load ratios (e.g. RIN compared
 403 to NEW, DUZ, POR and TAK). Consequently, the earthquake response velocity can be used as a parameter that
 404 defines the optimum solution. The following equation calculates the optimum slip load ratios for all types of
 405 earthquakes, giving an average error of 18% (better than both Equations 2 and 3) when compared with the
 406 results obtained for near and far-field natural earthquakes. This implies that the PGV factor can be a better
 407 parameter to estimate the optimum slip load values.

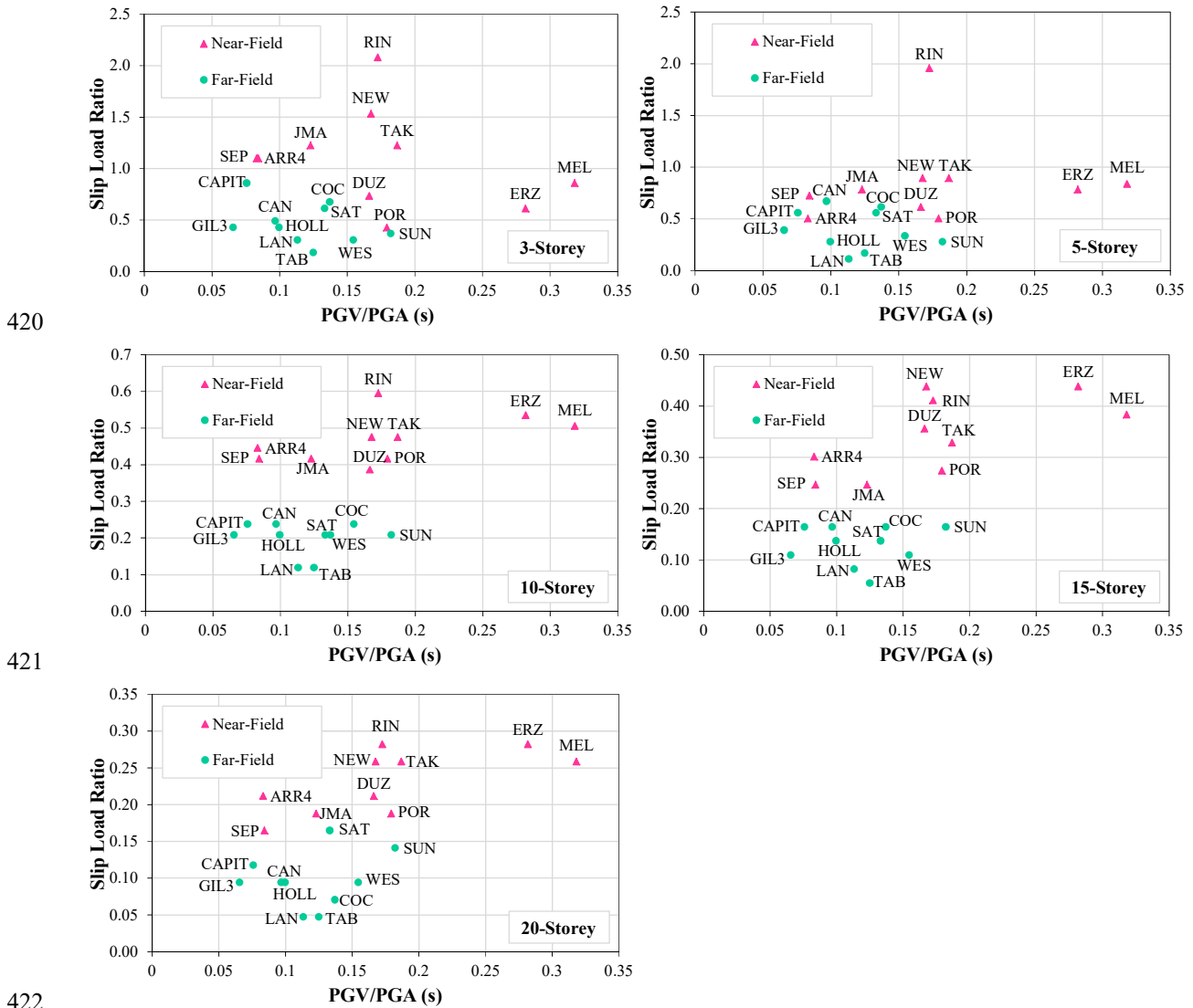
$$R_{EQ} = (4.75 \times e^{-0.09n} \times (a_v)^{0.75}) / 100 \quad (4)$$

408 where R_{EQ} is the optimum slip load ratio for both near-field and far-field earthquakes and a_v is the PGV of the
 409 earthquake.

410 Finally, by using a previously defined uniform cumulative pattern [Nabid et al., 2017] for the height-wise
 411 distribution of slip loads, the equation below can be used to find the slip load values at each storey level:

$$F_{S,i} = \frac{\sum_{j=i}^n F_{y,j} \times 4.75 \times e^{-0.09n} \times (a_v)^{0.75}}{100 \times n(n+1)/2} \times (n+1-i) \quad (5)$$

412 where $F_{s,i}$ and $F_{y,i}$ are the slip load and the storey shear strength of the i^{th} storey, respectively. It should be
 413 noted that the storey shear strength values can be calculated based on the results of non-linear pushover analysis.
 414 To avoid the effects of lateral load patterns, to obtain the shear strength of each storey, a single lateral load was
 415 applied at the same level, while the lateral degrees of freedom for all lower level storeys were constrained
 416 (Hajirasouliha and Doostan, 2010). The load-displacement curves were idealised by using bi-linear model
 417 proposed by ASCE/SEI 41-17, where the storey yield displacement is determined on the condition that the
 418 secant slope intersects the actual envelope curve at 60% of the nominal storey shear strength while the area
 419 enclosed by the bilinear curve up to failure point (here 4% drift) is equal to that enclosed by the original curve.

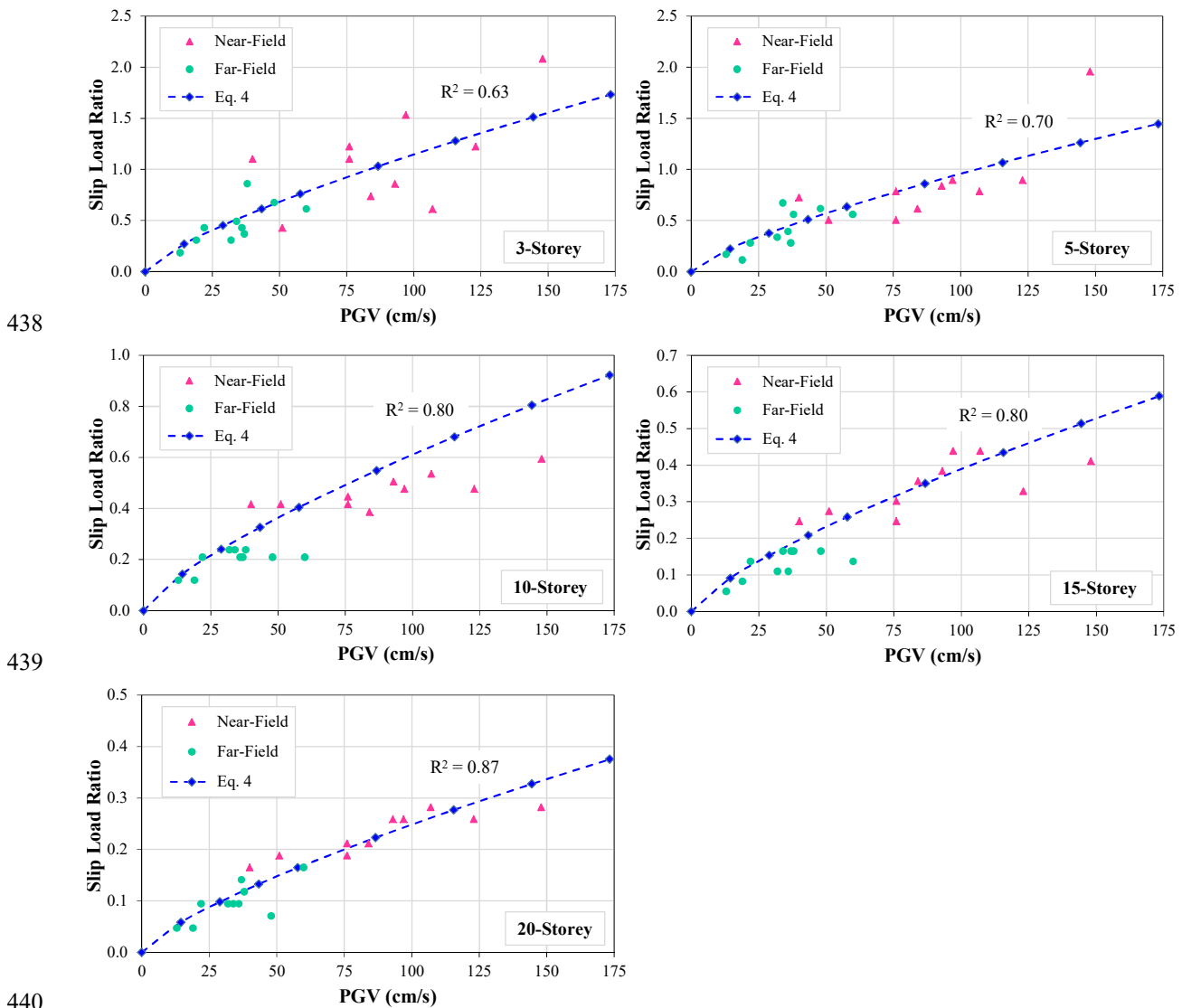


422
 423 **Figure 11.** Comparison of optimum slip load ratios of the 3, 5, 10, 15 and 20-storey frames under the selected
 424 near and far-field earthquakes as function of earthquake PGV/PGA ratio

425 The accuracy of the proposed empirical equation (Equation 4) can be assessed from Figure 12, showing the
 426 individual optimum slip load ratios obtained for the selected natural near-field and far-field earthquakes and the
 427 curves resulting from Equation 4 (as functions of earthquake PGV level). The proposed equation curve is the
 428 best fit to the series of optimum slip load ratios obtained for the selected earthquakes. The comparison with the
 429 results obtained with Equations 2 and 3, where PGA is the optimisation parameter (Figure 10), shows that PGV

430 is a more reliable parameter to determine the optimum design solutions for the frames subjected to both near and
 431 far-field earthquakes. For all the selected frames, the R-squared values corresponding to Equation 4 are
 432 significantly higher than those calculated for Equations 2 and 3, which confirms the higher accuracy of the new
 433 equation to predict the optimum slip load ratio under both near and far-field earthquakes.

434 It can be observed from Figure 12 that the upper parts of the data sets with higher optimum slip load ratios are
 435 associated with the results of the near-field earthquakes, whereas the lower parts correspond to those of the far-
 436 field records. The dispersion of the results and discrepancy between the data sets and the proposed equation
 437 curve can be caused by different pulse periods and frequency contents of the design earthquakes.



441 **Figure 12.** Comparison of optimum slip load ratios for the selected near and far-field earthquakes with the
 442 proposed empirical equation (Equation 4) as functions of earthquake PGV level

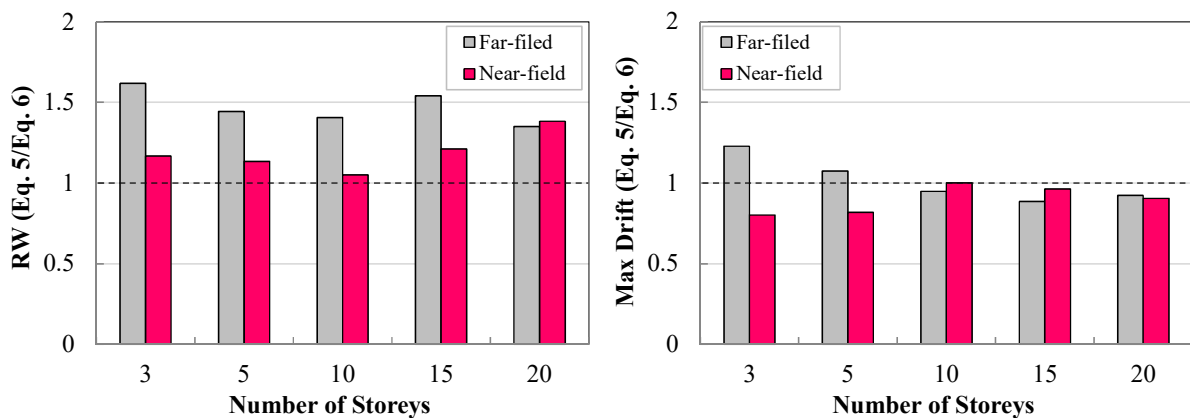
443 7. Efficiency of the Proposed Design Method

444 To assess the efficiency of the proposed design equation, the selected frames were designed using the slip load
 445 values obtained from Equation 5 and the following design equation suggested by Nabid et.al [2017]:

$$F_{S,i} = \frac{\sum_{i=1}^n F_{y,i} \times 1.12 \times e^{-0.11n}}{n(n+1)/2} \times (n+1-i) \quad (6)$$

446 The designed frames were then subjected to the selected near-field and far-field earthquakes. It should be noted
 447 that, unlike the equation proposed in this study, Equation (6) does not take into account the characteristics of the
 448 design earthquake (i.e. far-field and near-field effects). Figure 13 compares the average ratios between structural
 449 responses (i.e. energy dissipation parameters RW and maximum inter-storey drift) obtained by using Equation 5
 450 and Equation 6 for the 3, 5, 10, 15 and 20-storey frames subjected to the selected sets of near and far-field
 451 earthquakes. In general, the results indicate that the new design equation (Equation 5) increases the energy
 452 dissipation efficiency of the friction devices (i.e. average ratios above 1) and slightly decreases the maximum
 453 inter-storey drifts (i.e. average ratios below 1) of the studied frames. Based on the results, on average, the
 454 proposed design method could increase the energy dissipation efficiency parameters (RW) of the 3, 5, 10, 15
 455 and 20-storey frames by 17%, 13%, 5%, 21% and 38%, for the selected near-field records and by 62%, 44%,
 456 41%, 54% and 35%, for the far-field earthquakes, respectively. The maximum drift ratios (Equation 5 to
 457 Equation 6) are decreased by 20% and 11.4% for the near-field and far-field earthquakes, respectively. While
 458 more studies are required to assess the adequacy of the proposed empirical equations for the structures with
 459 geometries or structural systems different from those used in this study, the general design methodology
 460 proposed in this study should prove useful in preliminary design of friction dampers in practical applications.

461 It should be mentioned that the proposed friction wall system can be used in combination with the performance-
 462 based design methodology proposed by Montuori and Muscati (2016, 2017) to control the failure mechanism in
 463 the RC frames. Using this approach allows developing maximum number of dissipative zones at the beam ends,
 464 and hence improving the seismic performance of the system under strong earthquakes. The reliability of the
 465 design solutions can be also improved by using the partial safety factors related to the resistance model
 466 uncertainties in non-linear finite element analyses as proposed by Castaldo et al. (2018).



467
 468 **Figure 13.** Average ratios (this study to Nabid et al.'s [2017] study) of the (a) energy dissipation parameter
 469 (RW) and (b) maximum inter-storey drift for the 3, 5, 10, 15 and 20-storey frames under 20 near and far-field
 470 earthquakes

471 **8. Summary and Conclusion**

472 An efficient simplified model was proposed for optimum seismic design of friction-based dampers by
473 considering the effects of near-field and far-field ground motions. To obtain the optimum slip load ranges, a
474 comprehensive parametric study was performed on 3, 5, 10, 15, and 20-storey RC frames with friction wall
475 dampers under spectrum compatible earthquakes scaled to different PGA levels as well as a set of 20 near and
476 far-field earthquake records. Subsequently, empirical equations were proposed to obtain the optimum slip loads
477 based on the number of storeys and PGA (or PGV) of the design earthquake. The efficacy of the proposed
478 design equations in achieving maximum energy dissipation capacity was demonstrated under both near-field and
479 far-field earthquakes. Based on the results of this study, the following conclusions can be drawn:

- 480 • Higher PGA (or PGV) levels generally lead to lower energy dissipation efficiency with higher and wider
481 range of optimum slip load ratios. However, the relationship between the PGA and optimum slip load values
482 is not linear and depends on the number of storeys.
- 483 • Friction wall dampers exhibit, on average, 118% higher energy dissipation efficiency and 24% lower
484 maximum inter-storey drifts under far-field earthquakes compared to the near-field records. In general, the
485 optimum ranges of slip load ratios obtained for the frames under the near-field earthquakes were also
486 noticeably wider and higher (about 1.5 times) compared to those achieved under the far-field ground
487 motions.
- 488 • It was shown that for the same PGV/PGA level (or similar frequency content), the earthquakes with higher
489 PGA and PGV values resulted in higher optimum slip load ratios. In addition, the earthquakes with relatively
490 high velocities occurring at the periods close to the period of the corresponding bare frames result in higher
491 range of optimum slip load values.
- 492 • The optimum response of the structures was more sensitive to the variation of PGV than PGA. The proposed
493 design equation for optimum slip load ratio R as a function of PGV resulted in considerably lower
494 dispersions of the results (i.e. higher R-squared values) compared to the equations using PGA as a design
495 variable.
- 496 • Compared to the previous equation suggested by Nabid et al. [2017] (without consideration of far-field/
497 near-field effects), the design method proposed here is considerably more efficient in increasing the energy
498 dissipation efficiency of the friction devices (up to 54%) and decreasing the maximum inter-storey drift of
499 the studied frames (up to 20%).

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