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Nabid, N., Hajirasouliha, I. and Petkovski, M. (2017) A practical method for optimum seismic design of friction wall dampers. *Earthquake Spectra*, 33 (3). pp. 1033-1052.

<https://doi.org/10.1193/110316EQS190M>

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A Practical Method for Optimum Seismic Design of Friction Wall Dampers

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Friction control systems have been widely used as one of the efficient and cost effective solutions to control structural damage during strong earthquakes. However, the height-wise distribution of slip loads can significantly affect the seismic performance of the strengthened frames. In this study, a practical design methodology is developed for more efficient design of friction wall dampers by performing extensive nonlinear dynamic analyses on 3, 5, 10, 15, and 20-story RC frames subjected to seven spectrum-compatible design earthquakes and five different slip load distribution patterns. The results show that a uniform cumulative distribution can provide considerably higher energy dissipation capacity than the commonly used uniform slip load pattern. It is also proved that for a set of design earthquakes, there is an optimum range for slip loads that is a function of number of stories. Based on the results of this study, an empirical equation is proposed to calculate a more efficient slip load distribution of friction wall dampers for practical applications. The efficiency of the proposed method is demonstrated through several design examples.

INTRODUCTION

Much of the existing building structures in developing countries are designed primarily to sustain gravity loads with little or no seismic detailing. Many catastrophic failures in RC buildings during recent major earthquakes (e.g. Kashmir, 2005; China, 2008; Indonesia, 2009; Haiti, 2010; Turkey, 2011; Nepal, 2015) have highlighted the urgent need to improve the seismic performance of these substandard buildings. Passive energy dissipation devices have been proven as one of the most efficient and cost effective solutions in terms of controlling structural damage during strong earthquakes by dissipating the imparted seismic energy and reducing damage in structural elements (Symans et al., 2008; Soong and

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28 Costantinou, 2014). Among the different types of passive energy dissipation devices, friction-
29 based dampers usually have the highest energy dissipation capacity for the same levels of
30 force and deformation (Pall and Pall, 2004). Moreover, friction devices are in general
31 velocity and temperature-independent, can be easily tuned to the characteristics of the
32 structure, and provide sustained performance under large number of cycles (Grigorian et al.,
33 1993, Aiken et al., 1993, Pall and Pall, 2004).

34 Pall and Marsh (1982) introduced the first generation of friction dampers for braced steel
35 frames, which were designed to slip under a predetermined load before the buckling of the
36 braces occurred. Wu et al. (2005) developed an improved model of Pall friction dampers
37 using a T-shaped core plate, which was easier to manufacture and assembly. Slotted Bolted
38 Connections (SBC) were initially used by Fitzgerald (1989) to dissipate earthquake input
39 energy and prevent buckling of brace elements in steel braced frames. The energy absorbing
40 mechanism in SBCs is based on the friction between the gusset plates and the sliding
41 channels. More recently, shear slotted bolted connections (SSBC) were proposed to extend
42 the application of SBC in members with shear-dominated behavior (Nikoukalam et al., 2015).

43 While most of existing friction-based dampers were developed for steel bracing systems,
44 using brace elements in RC frames can lead to high stress concentration and damage in the
45 connection zones. This problem can be addressed by using wall-type systems that provide
46 enough space to transfer lateral forces to the adjacent elements. Sasani and Popov (1997)
47 experimentally and analytically investigated the performance of a wall-type friction damper
48 using lightweight concrete panels. Their proposed system consisted of a precast concrete wall
49 which was connected to the lower floor beam by bolted supports and to the upper floor beam
50 by friction energy dissipating connectors. In a follow up study, they increased the efficiency
51 of their proposed system by using epoxy-anchored bolts to provide adequate strength and
52 stiffness at the base supports to minimize the rocking movement of the wall panels during
53 strong earthquakes (Sasani and Popov, 2001). Petkovski and Waldron (2003) studied the
54 effectiveness of friction-based concrete wall dampers (with and without opening) to improve
55 the seismic performance of 6, 8 and 10-story RC structures subjected to four real earthquake
56 records. They concluded that, irrespective of the stiffness of the wall panels, there was an
57 optimum range for the slip force in the friction connections that led to the best seismic
58 performance. Although their proposed friction wall dampers were designed not to transfer
59 additional shear forces to the adjacent columns, the results of their study showed that they

60 still considerably increase the base shear and the axial loads of the columns. However, these
61 adverse effects can be controlled by limiting the slip forces in the friction dampers as it will
62 be discussed in this study. A similar wall friction damper was proposed by Cho and Kwon
63 (2004), incorporated an RC wall connected to the upper floor beam using a T-shape steel
64 device with Teflon sliding sheets. In their system, the clamping force could be easily adjusted
65 based on the expected earthquake magnitude using an oil jack loading system.

66 While several research studies have covered the optimum design of viscous and
67 viscoelastic dampers (e.g. Park et al. 2004, Levy and Lavan 2006, Takewaki 2011, Whittle et
68 al. 2012, Adachi et al. 2013, Sonmez et al. 2013), very limited studies are focused on the
69 optimization of friction-based dampers subjected to seismic actions. In one of the early
70 attempts, Filiatrault and Cherry (1990) proposed a simplified seismic design procedure to
71 obtain the optimum slip load values by minimizing an energy derivation parameter denoted
72 as relative performance index (RPI). It was shown that the optimum slip load values depend
73 more on the amplitude and frequency of the design earthquake rather than the structural
74 characteristics. Subsequently, Moreschi and Singh (2003) used Genetic Algorithm (GA) to
75 determine the optimum height-wise placement of yielding metallic and friction dampers in
76 braced steel frames. Patro and Sinha (2010) investigated the seismic performance of shear-
77 frame building structures with dry-friction devices, using uniform height-wise slip load
78 distribution. They showed that, in general, a suitable slip load range can be determined such
79 that the seismic response of the structure is nearly optimal for a wide range of ground motion
80 characteristics. Fallah and Honarparast (2013) optimized the slip load distribution and
81 placement of Pall friction dampers in multi-story shear braced frame using a non-dominated
82 sorting genetic algorithm (NSGA-II). In a more recent study, Miguel et al. (2016) adopted a
83 backtracking search optimization algorithm to simultaneously optimize the location and slip
84 load distribution of friction dampers subjected to seismic loading.

85 It should be noted that most of the above mentioned optimization techniques may not be
86 suitable for practical design purposes due to the high computational efforts required to
87 analyze a large number of non-linear dynamic systems. This study aims to develop, for the
88 first time, a practical method for more efficient design of friction-based wall dampers under
89 earthquake loads without using complex optimization techniques. To obtain the best slip load
90 distribution along the height of the building, extensive nonlinear dynamic analyses are
91 conducted on 3, 5, 10, 15, and 20-story RC frames subjected to a set of earthquake records

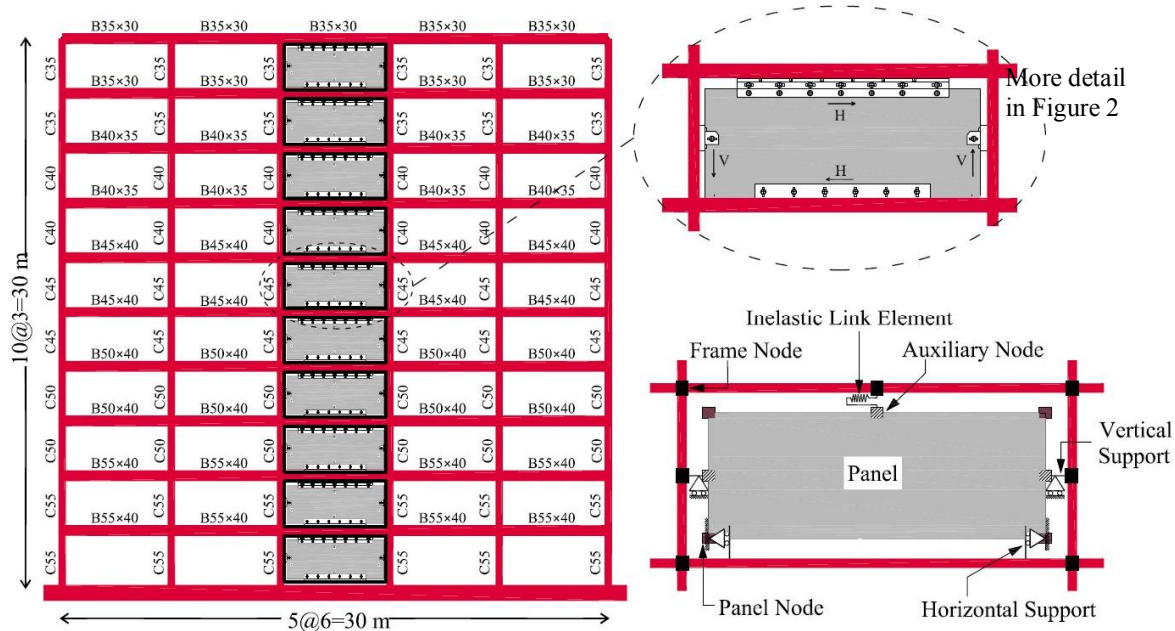
92 representing a design spectrum. The results are then used to develop an empirical design
93 equation, which leads to design solutions with maximum energy dissipation in the friction
94 wall dampers. The efficiency of the proposed equation is demonstrated through several
95 design examples.

96 **MODELING AND ASSUMPTIONS**

97 **REFERENCE FRAMES**

98 In this study 3, 5, 10, 15 and 20-story RC frames were selected with the typical geometry
99 shown in Figure 1. The frames were assumed to be located on a soil type D of the IBC (2015)
100 category, with the design spectral response acceleration at short periods and 1-sec period
101 equal to 0.40g and 0.64g, respectively. To represent substandard RC structures, the frames
102 were designed based on the low-to-medium seismicity regions using a design earthquake
103 with PGA of 0.2g. The uniformly distributed dead and live loads were assumed as 6 kN/m²
104 and 2 kN/m² for interior stories, and 5 kN/m² and 1.5 kN/m² for the roof level. The frames
105 were designed to support the seismic loads based on IBC (2015) and ASCE/SEI 7-10 (2010)
106 and in accordance with the minimum requirements of ACI 318 (2014) for RC frames with
107 intermediate ductility. The concrete compressive strength (f'_c) and the yield strength of steel
108 reinforcement bars (f_y) were assumed to be 35 and 400 Mpa, respectively. Square and
109 rectangular sections were used for column and beam elements as shown in Figure 1 for the
110 10-story frame.

111 To predict the seismic response of the RC frames, nonlinear time-history analyses were
112 carried out using computer program DRAIN-2DX (Prakash et al. 1993). Rayleigh damping
113 model with a constant damping ratio of 0.05 was assigned to the first mode and to any mode
114 at which the cumulative mass participation exceeded 95%. Nonlinear moment-rotation (M- θ)
115 and axial-moment (P-M) plastic hinges were assigned at both ends of RC beam and column
116 elements, respectively, using element Type 2 in DRAIN-2DX. The friction mechanism at the
117 top edge of the panel was modeled by means of an inelastic link element (element Type 4 in
118 DRAIN-2DX) to provide an ideal Coulomb friction hysteretic behavior. In this study, it was
119 assumed that the strength of the concrete wall panel is always greater than the effects of the
120 maximum slip load of the friction device. Therefore, the wall panels were modeled with
121 elastic panel elements (15 cm thickness) using element Type 6 in DRAIN-2DX. To consider
122 rigid diaphragms in the analytical models, the frames nodes were constrained to each other in
123 horizontal direction.



124

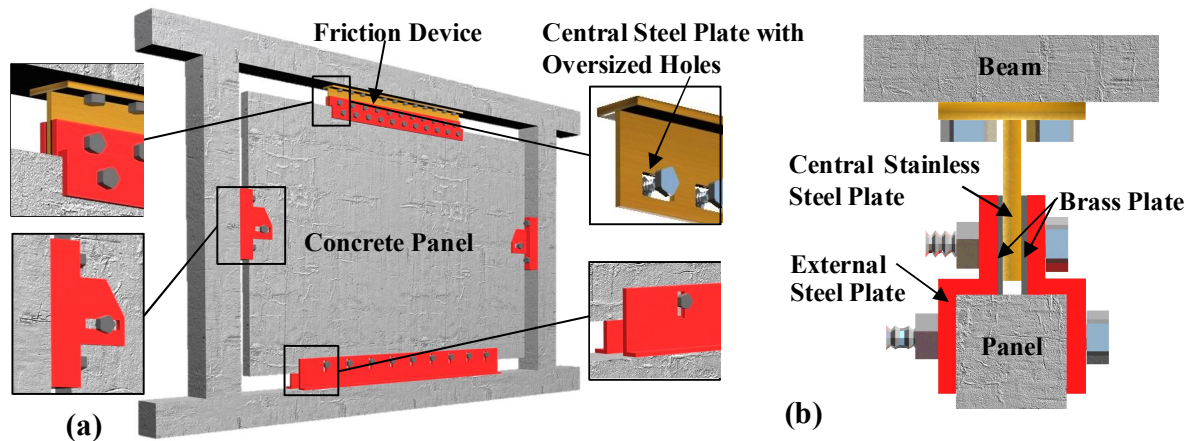
125 **Figure 1.** Schematic geometry of the reference RC frames and the analytical model of the studied
 126 friction-based wall dampers

127 **PROPOSED FRICTION-BASED WALL DAMPER**

128 The friction-based wall damper used in this study consists of a structural concrete panel
 129 that is connected to the frame by using two vertical supports in the sides, one horizontal
 130 connection at the bottom, and a friction device at the top. Figure 2 illustrates the details of the
 131 proposed friction panel. The vertical support for the concrete panel is provided by using
 132 panel-to-column connections with horizontal slots, which prevent transfer of shear forces to
 133 the columns. The panel is connected to the lower floor by horizontally fixed connections with
 134 vertical slots to avoid transferring shear forces to the beams. This arrangement will ensure
 135 that the displacement of the friction device at the top of the panel is equal to the inter-story
 136 drift at each level. The proposed friction device is a simple panel-to-frame Slotted Bolted
 137 Connection, which consists of two steel plates bolted at the top of the panel (external plates)
 138 clamped together over a slotted stainless steel plate anchored to the top beam (central plate).
 139 The friction mechanism is obtained through friction between the central stainless steel plate
 140 and the two brass plates (see Figure 2 (b)). Extensive experimental tests conducted by
 141 Grigorian et al. (1993) demonstrated the reliable hysteretic behavior of this type of friction
 142 device under sinusoidal and simulated seismic imposed displacements.

143 By using over-sized holes in the central steel plate (as shown in Figure 2), the largest
 144 friction forces will occur between the central and the brass plates. The size of these holes in
 145 the horizontal and vertical directions can be calculated to accommodate the expected

146 maximum lateral drift and vertical deformations of the beam, which would prevent transfer of
 147 large stresses on the central plate around the slotted holes. The concentrated moments applied
 148 to the columns at the location of the connections should be considered in the design process
 149 of the proposed friction wall system. The results of this study indicate that these additional
 150 loads are relatively low compared to the maximum bending moments in the corresponding
 151 bare frame.

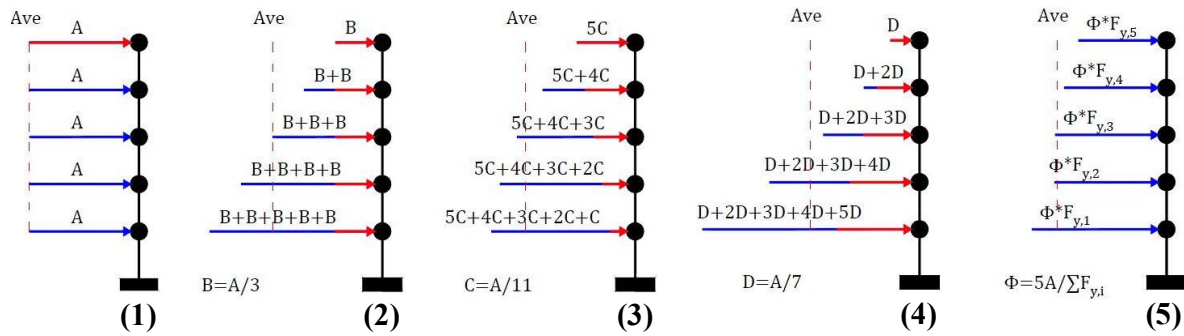


152 (a) 153 **Figure 2.** Schematic view of the (a) proposed friction wall damper, (b) friction device

154 **SLIP LOAD DISTRIBUTION PATTERNS**

155 The slip force in the friction connections of the proposed wall damper can be adjusted and
 156 tuned independently for each story by controlling the clamping forces of the bolts. Such
 157 capability provides the possibility of using the same connection with optimized slip loads.
 158 Wall dampers with very low slip loads (i.e. $F_s \cong 0$) do not have any lateral load resistance
 159 and, therefore, are not considered as structural elements. On the contrary, using large slip
 160 load values may lead to a connection lock-up under design earthquakes, which implies the
 161 passive control system behaves as a fixed wall panel with negligible energy dissipation
 162 capacity. In practical applications, a uniform height-wise slip load distribution is usually
 163 employed for design of passive friction dampers. However, this may not necessarily lead to
 164 an optimum design solution for a range of structures and design earthquakes.

165 To identify more efficient slip load distributions, five different distribution patterns are
 166 considered: (1) uniform, (2) uniform cumulative, (3) triangular cumulative, (4) inverted
 167 triangular cumulative and (5) a distribution proportional to the story shear strengths. Figure 3
 168 shows the different slip load distribution patterns, scaled to produce the same base shear in
 169 first mode response (i.e. $\Sigma F_s = \text{constant}$). The shear strength of each story ($F_{y,i}$) can be
 170 calculated from a non-linear pushover analysis (Hajirasouliha and Doostan, 2010).



171

172 **Figure 3.** Typical patterns of the selected slip load distributions with the same average value

173 **SELECTED SEISMIC EXCITATIONS**

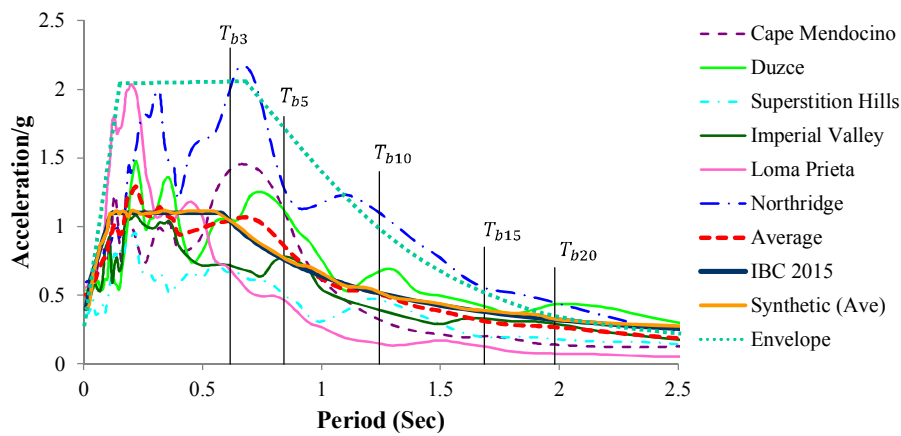
174 The reference structures are subjected to six real strong ground motions: Cape Mendocino
 175 1992, Duzce 1999, Superstition Hills 1987, Imperial Valley 1979, Loma Prieta 1989, and
 176 Northridge 1994. The characteristics of the selected records are listed in Table 1. All of these
 177 ground motions correspond to soil class D of IBC-2015 and are recorded in low to moderate
 178 distances from the epicenter (less than 45 km) with high local magnitudes (i.e. $M > 6.5$).

179 **Table 1.** Characteristics of the selected seismic excitation records

No.	Earthquake Name	M	Record	Duration (s)	PGA (g)	PGV (Cm/s)	PGD (Cm)
1	1992 Cape Mendocino	6.9	CAPEMEND/PET000	36	0.590	48.4	21.74
2	1999 Duzce, Turkey	7.2	DUZCE/DZC270	26	0.535	83.5	51.59
3	1987 Superstition Hills (B)	6.7	SUPERST/B-ICC000	60	0.358	46.4	17.50
4	1979 Imperial Valley	6.5	IMPVALL/H-E04140	39	0.485	37.4	20.23
5	1989 Loma Prieta	6.9	LOMAP/G03000	40	0.555	35.7	8.21
6	1994 Northridge	6.7	NORTHR/NWH360	40	0.590	97.2	38.05

180 Figure 4 illustrates the 5% damped elastic acceleration response spectra of the six natural
 181 earthquake records in Table 1. It is shown that, on average, the selected ground motions
 182 provide a close approximation to the design response spectra of IBC-2015 for the site class D
 183 in high seismic zones (i.e. $PGA = 0.4g$). This is particularly evident at the first mode periods
 184 of the bare frames denoted as T_{b3} to T_{b20} . Therefore, in this study these earthquake records
 185 are used directly without being normalized. A set of five synthetic earthquake records with a
 186 PGA of $0.4g$ is also generated using SIMQKE program (Vanmarke, 1976) to be compatible
 187 with the soil type D of IBC (2015) elastic design spectrum. To simulate non-stationary
 188 spatially variable ground motions, a trapezoidal intensity envelope function with the rise
 189 time, level time and total duration of 2.5, 12 and 35 sec, respectively, was applied. Figure 4
 190 demonstrates a good compatibility between the average spectrum of the synthetic earthquakes

191 and the IBC (2015) design spectrum. Therefore, these synthetic earthquakes can be
 192 considered to be good representatives of the design response spectrum.



193
 194 **Figure 4.** Comparison between elastic spectral acceleration of the six selected earthquakes, average of
 195 five synthetic earthquakes and IBC-2015 design spectrum for soil type D, 5% damping ratio. T_{b3} to
 196 T_{b20} are first mode periods of the bare frames

197 RC FRAMES WITH FRICTION-BASED WALL DAMPERS

198 To investigate the efficiency of the proposed passive-control system, a wide range of slip
 199 load values and height-wise distribution patterns are considered, aiming to cover all practical
 200 design solutions. Different structural performance parameters such as maximum inter-story
 201 drift, roof displacement, maximum axial load in columns, base shear, and cumulative energy
 202 dissipation are calculated. For comparison purposes, the slip load ratio F_{SR} is defined as:

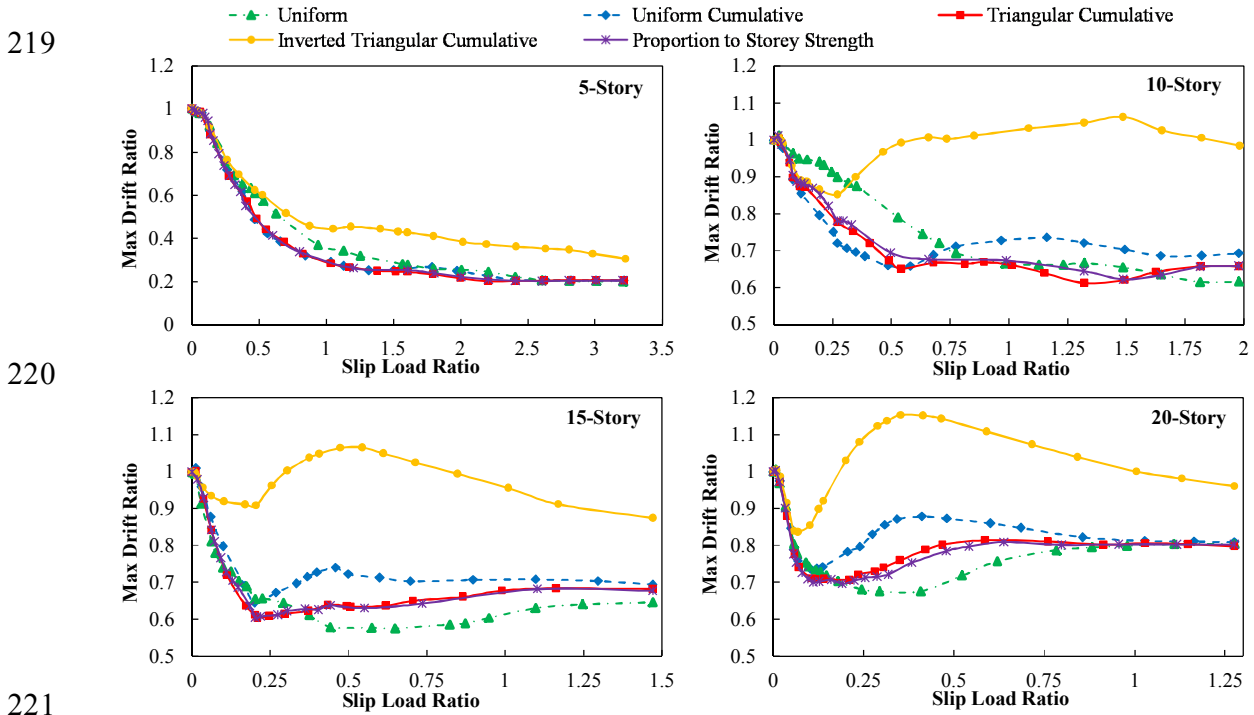
$$203 \quad F_{SR} = \frac{\sum_{i=1}^n F_{s,i}}{\sum_{i=1}^n F_{y,i}} \quad (1)$$

204 where n is number of stories, $F_{s,i}$ is slip force at i^{th} story, and $F_{y,i}$ is story shear strength
 205 of the i^{th} story. Using this parameter helps to compare the effects of using different slip load
 206 distributions, while the base shear force remains constant.

207 MAXIMUM INTER-STORY DRIFT

208 Maximum inter-story drift is widely used to evaluate the level of damage to both
 209 structural and non-structural elements in RC structures (Hajirasouliha et al. 2012). Figure 5
 210 shows the variation of maximum inter-story drift ratios (normalized to the bare frames) for 5,
 211 10, 15 and 20-story frames using five different slip load distribution patterns with a wide
 212 range of slip load ratios F_{SR} . The results are the average of the displacement demands
 213 obtained in the six selected earthquakes listed in Table 1. The energy dissipation capacity of

214 wall panels with very small F_{SR} values is negligible, and therefore, their response is close to
 215 that of bare frames (normalized response parameters are close to 1.0). Figure 5 demonstrates
 216 a similar trend for different slip load patterns, where the maximum drift ratios generally
 217 reduce by increasing the friction slip load ratios up to a certain limit. This is followed by a
 218 constant trend in 3 and 5-story and an ascending trend in 10, 15 and 20-story frames.



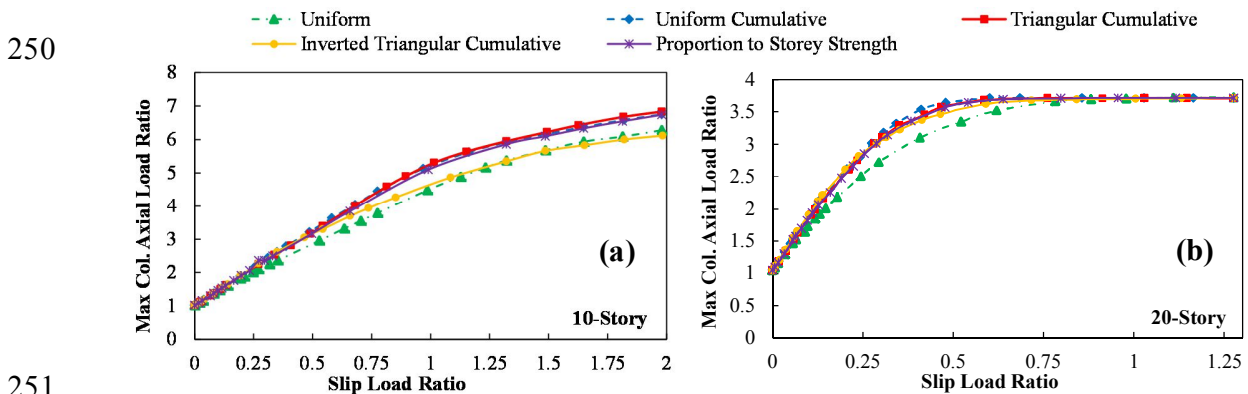
222 **Figure 5.** Variation of maximum inter-story drift for 5, 10, 15 and 20-story RC frames using different
 223 slip load distributions, average of the six selected earthquakes

224 The results in Figure 5 indicate that there is an optimum range for slip load ratios that, on
 225 average, leads to lower inter-story drifts. Similar conclusions have been reported by
 226 Petkovski and Waldron (2003) and Fallah and Honarparast (2013) for other types of friction
 227 dampers. Figure 5 shows that by using friction wall dampers with more efficient slip load
 228 distributions, the maximum inter-story drift of 3, 5, 10, 15, and 20-story frames reduced by
 229 up to 85%, 75%, 38%, 40%, and 30%, respectively. This implies that the reduction in
 230 maximum drift ratio is more prominent in low rise buildings. While the inverted triangular
 231 cumulative slip load distribution (Type 4 in Figure 3) seems to be less effective in reducing
 232 maximum inter-story drifts, other distribution patterns lead to similar levels of reduction.

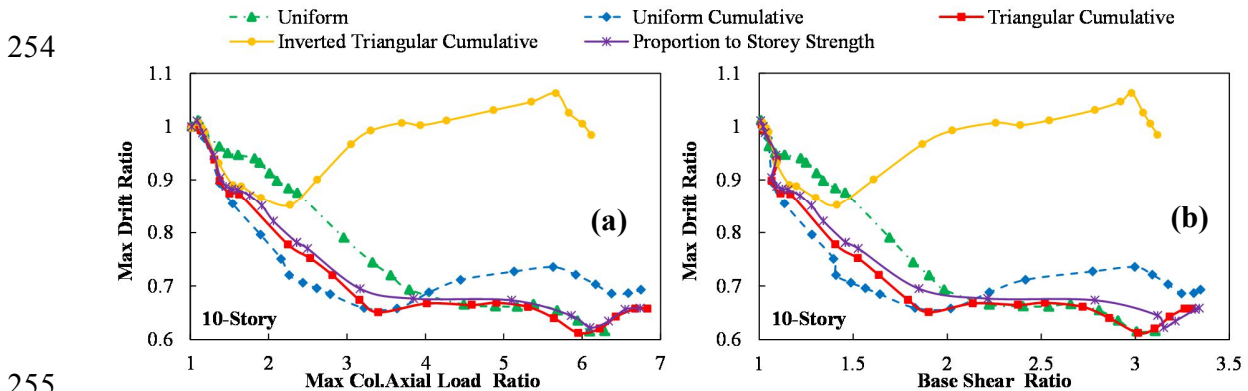
233 **COLUMN AXIAL LOAD**

234 Figures 6 (a and b) display the maximum axial load ratios (normalized to the bare frames)
 235 of the columns connected to the friction wall dampers in 10 and 20-story frames using

236 different slip load ratios. The results show that, regardless of the selected slip load
 237 distribution pattern, the maximum axial load in the columns increases by increasing the slip
 238 load ratios up to a steady-state level (see Figure 6 (b)). At this stage, the wall dampers are
 239 locked at all story levels, which is referred to as “fixed-wall” in this study. As expected,
 240 increasing slip load ratios beyond this limit does not affect the seismic performance of the
 241 frames. It is shown that, for the same slip load ratio, using uniform distribution (Type 1 in
 242 Figure 3) results in lower axial loads compared to other slip load distributions. However, for
 243 practical design purposes, it is important to obtain slip load ratios that control the lateral
 244 displacement demands of the structure without imposing high axial loads to the columns and
 245 foundations. Figures 7 (a and b) compare the maximum column axial load ratio for different
 246 slip load distributions as a function of maximum inter-story drift. The results in general
 247 indicate that, for a specific inter-story drift, using a uniform cumulative distribution (Type 2
 248 in Figure 3) leads to minimum axial loads compared to other slip load distributions. A similar
 249 trend was observed for the other frames with different number of stories.



251
 252 **Figure 6.** Variation of maximum column axial load ratio as a function of slip load ratio for (a) 10 and
 253 (b) 20-story frame, average of the six selected earthquakes

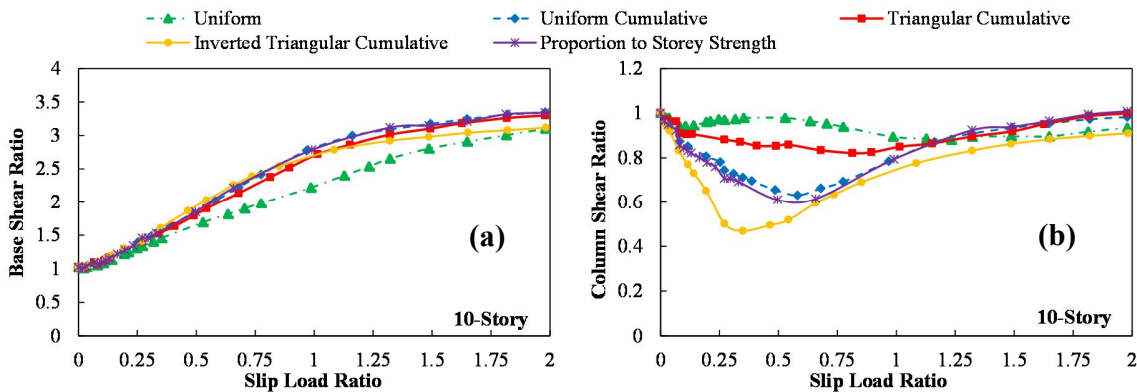


255
 256 **Figure 7.** Variation of maximum drift ratio as a function of (a) column axial load and (b) base shear
 257 ratio for 10-story frame, average of the six selected earthquakes

258 **BASE SHEAR**

259 Increasing the base shear demand is one of the main barriers to the use of passive control
 260 systems such as shear walls and bracings. Although the proposed friction wall damper
 261 increases the base shear demand of the bare frame, this increase can be efficiently controlled
 262 by using appropriate slip loads in friction devices. For example, Figure 8 (a) compares the
 263 base shear and the maximum column shear force ratios of 10-story frames with different slip
 264 load distributions as a function of the slip load ratio. The results show that increasing the slip
 265 loads is always accompanied by an increase of the base shear until a maximum level is
 266 reached. For similar slip load ratios, using uniform slip load distribution leads to lower base
 267 shear when compared with other distribution patterns. However, for the same inter-story drift
 268 ratios, uniform cumulative slip load distribution in general leads to lower base shear values
 269 compared to the other distribution patterns (see Figure 7 (b)).

270



271

272 **Figure 8.** Variation of: (a) maximum column shear force ratio, and (b) base shear ratio as functions of
 273 slip load ratio, 10-story frame, average of the six selected earthquakes

274 It should be noted that the proposed friction wall damper is capable of transferring some
 275 of the base shear forces directly to the foundation at the ground floor. Therefore, despite
 276 increasing the total base shear, the proposed wall dampers can generally reduce the maximum
 277 shear forces in the columns at the base of the structure. For instance, the results in Figure 8
 278 (b) indicate that unlike the base shear, increasing the slip load ratio is usually accompanied
 279 by a decrease in the maximum column shear forces until a minimum value is reached.

280 The most reduction in the maximum column shear forces was observed in the frame with
 281 the inverted triangular cumulative pattern (Type 4 in Figure 3). The main reason is that, for
 282 the same average slip load, the inverted triangular pattern has larger slip load values at the
 283 ground floor. This implies that the friction wall system can transfer higher shear forces
 284 directly to the foundation, which reduces the maximum shear forces at the columns.

285 **ENERGY DISSIPATION CAPACITY**

286 In this study, R_{w1} is defined as the ratio of the deformation work of structural elements in
287 the structure with friction wall dampers (W_{cs}) to that in the corresponding bare frame (W_{bf}):

288
$$R_{w1} = \frac{W_{cs}}{W_{bf}} = \frac{(W_{sb} + W_{sc})_{cs}}{(W_{sb} + W_{sc})_{bf}} \quad (2)$$

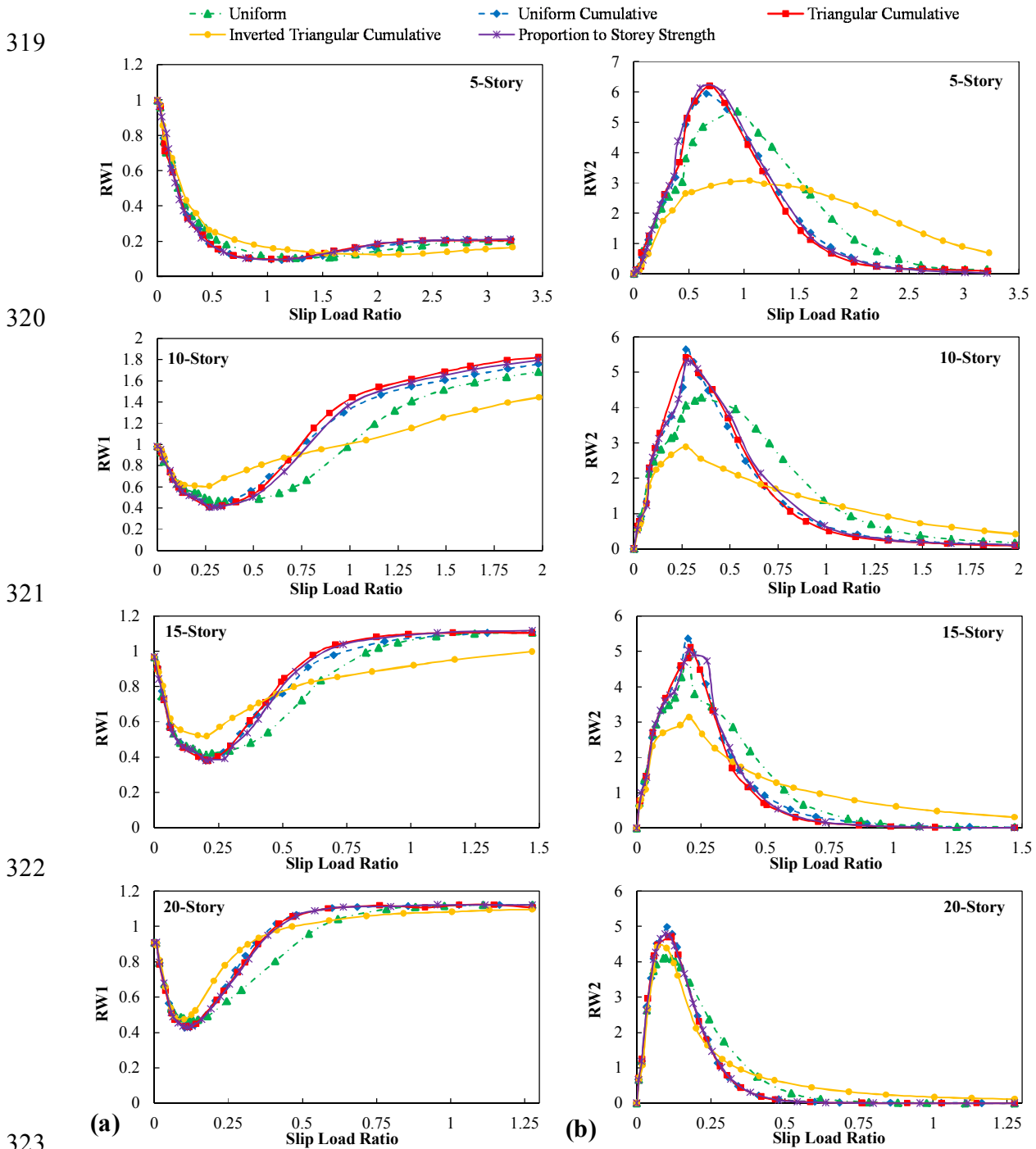
289 where W_{sb} and W_{sc} denote the static work of the beam and column elements,
290 respectively. R_{w1} decreases by increasing the efficiency of the friction wall dampers in
291 dissipating the earthquake input energy. Figure 9 (a) shows the R_{w1} as a function of the slip
292 load ratio for 3, 5, 10, 15 and 20-story frames using different slip load distribution patterns.
293 In general, R_{w1} reaches a minimum value at a slip load ratio which is almost independent of
294 the selected slip load distribution pattern. This implies that there is an optimum range for the
295 slip load ratios that leads to the lowest deformation work (or structural damage) in the
296 structural elements. The reduction in R_{w1} is more evident in low- to medium-rise buildings.
297 The results also indicate that the optimum slip force ratios decrease by increasing the number
298 of stories (from $F_{SR}=1$ in 3-story to $F_{SR}=0.15$ in 20-story frames). Also it can be noted that,
299 in general, the optimum range narrows by increasing the number of stories.

300 The amount of energy dissipated in the friction device under a design earthquake can be
301 evaluated by calculating the ratio of the friction work in the wall dampers (W_{sf}) to the
302 deformation work of the main structural elements (W_{cs}):

303
$$R_{w2} = \frac{(W_{sf})_{cs}}{(W_{sb} + W_{sc})_{cs}} \quad (3)$$

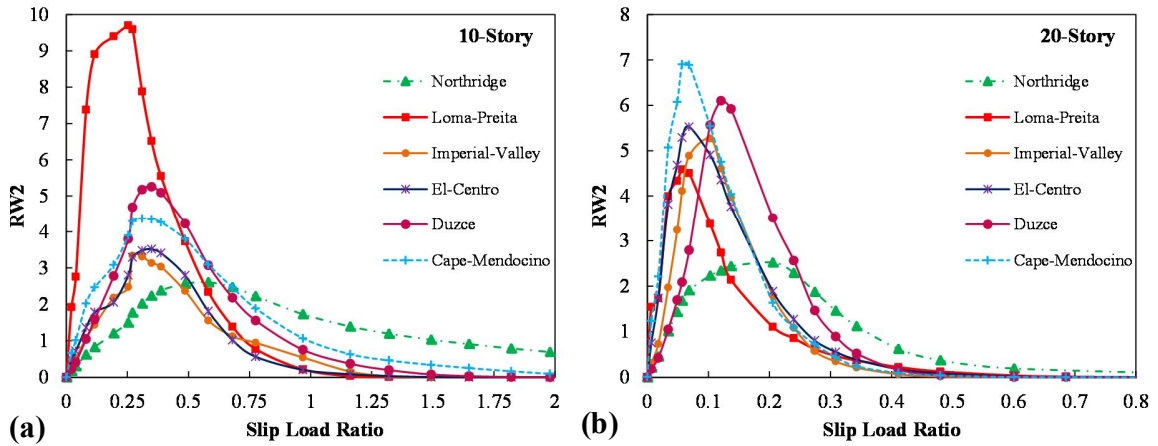
304 While R_{w1} gives a measure of the efficiency of the dampers in reducing the energy
305 dissipation demand of the structural elements, R_{w2} represents the energy dissipation capacity
306 of the dampers. The variation of R_{w2} as a function of the slip load ratio is illustrated in
307 Figure 9 (b) for 3, 5, 10, 15 and 20-story frames. The R_{w2} parameter tends to zero for very
308 low and very high slip forces. The reason is that the energy dissipated in the dampers with
309 very low slip forces is negligible, while the dampers with very high slip forces are locked and
310 hence do not dissipate any energy. The results indicate that the overall trend of R_{w2} is similar
311 for all the reference frames irrespective to the number of stories. However, on average, by
312 increasing the number of stories the maximum R_{w2} values are reached at lower slip load
313 ratios. It is evident that the uniform cumulative slip load pattern is usually the most effective

314 pattern in terms of increasing the energy dissipation capacity of the friction-based wall
 315 dampers (except for the 3-story frame), while the inverted triangular cumulative pattern is the
 316 least efficient. Based on the results in Figure 9, the optimum range of the slip load ratios for
 317 3, 5, 10, 15, and 20-story frames with uniform cumulative slip load distribution is within
 318 0.65-0.95, 0.55-0.85, 0.25-0.45, 0.10-0.30, and 0.05-0.15, respectively.



324 **Figure 9.** Envelope of energy dissipation parameters (a) R_{w1} and (b) R_{w2} as a function of the slip
 325 load ratio, average of the six selected earthquakes

326 Figure 10 shows the variation of energy dissipation parameter R_{w2} as a function of the slip
 327 load ratio for the 10-story and 20-story frames subjected to the six selected real excitation
 328 records. It is evident that the amount of energy dissipated in the wall dampers is highly
 329 dependent on the input earthquake and the slip load ratio. However, the results show that the
 330 range in which the slip load ratio R_{w2} reaches maximum (i.e. the best damper performance)
 331 is not significantly affected by the selected design earthquake. This conclusion was
 332 confirmed by the results for all the reference frames.



333 (a) 334 **Figure 10.** Envelope of R_{w2} energy dissipation parameter for (a) 10-story frame, (b) 20-story frame
 335 as a function of the slip load ratio, selected real earthquakes

336 A PRACTICAL DESIGN METHOD FOR FRICTION WALL DAMPERS

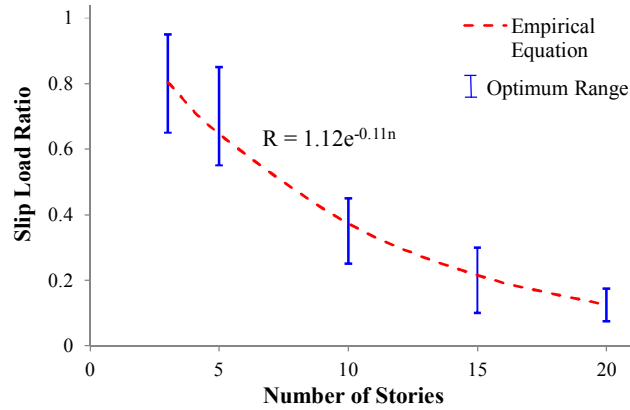
337 Figure 11 shows the optimum range of the slip load ratios obtained in the previous section
 338 as a function of number of stories. The optimum design solutions for low rise buildings tend
 339 to a fixed wall system, while for high-rise buildings the best design solutions have lower
 340 average slip load ratios. It is shown in Figure 11 that the average value of the optimum slip
 341 load ratios can be represented by the following exponential function:

$$342 \quad R = 1.12e^{-0.11n} \quad (4)$$

343 where R is the most appropriate slip load ratio and n is the number of stories. The slip
 344 load ratio R calculated from Equation 4 is the ratio between the average of the slip loads with
 345 uniform cumulative distribution and the average of the story shear strengths. Therefore, the
 346 following equation can be used to acquire the more efficient slip load values at each story:

$$347 \quad F_{si} = \frac{\sum V_{si} \times R}{n(n+1)/2} \times (n+1-i) = \frac{\sum V_{si} \times 1.12e^{-0.11n}}{n(n+1)/2} \times (n+1-i) \quad (5)$$

348 where n is the number of stories; and F_{si} and V_{si} are the slip load and the story shear. It
 349 should be noted that Equation 4 is based on the models considered in this study, and the
 350 optimum range might change for the structures with other dynamic characteristics.



351
 352 **Figure 11.** Comparison between the empirical equation and the best analytical slip load range for
 353 frames with different number of stories

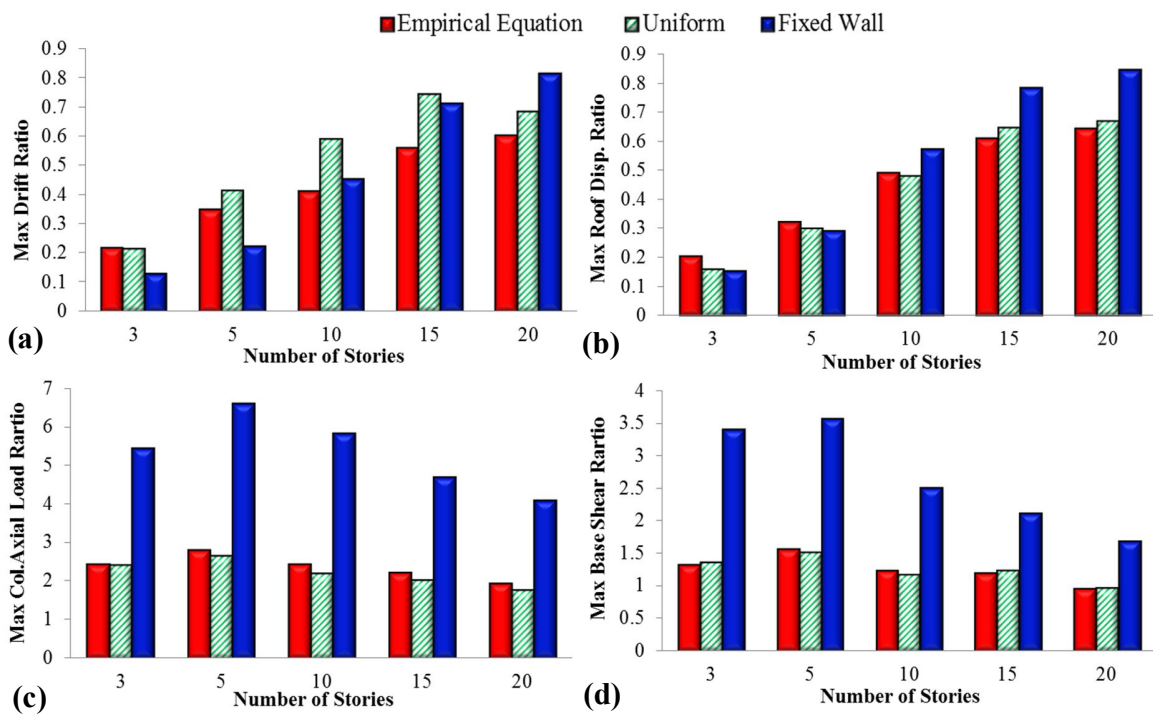
354 EFFICIENCY OF THE PROPOSED PRACTICAL DESIGN METHOD

355 The efficiency of the proposed equation to obtain more efficient design solutions is
 356 investigated for 3, 5, 10, 15, and 20-story frames under a set of five design spectrum
 357 compatible synthetic earthquakes (see Figure 4). For comparison purposes, the seismic
 358 performances of the frames with friction wall dampers designed using Equation 4 are
 359 compared with those designed based on the uniform slip load distribution (i.e. conventional
 360 design) as well as the frames with fixed panel-to-frame connections. The more efficient slip
 361 load values at different stories are calculated by using Equation 5. For a better comparison,
 362 the slip load values are scaled in the frames with uniform slip load distribution (without
 363 changing the distribution pattern) to have a similar average value in all design solutions.

364 Figure 12 shows that, in general, the friction-based wall dampers designed with the
 365 proposed slip load distribution pattern provide better design solutions with lower maximum
 366 drift and roof displacement ratios compared to the conventionally designed wall dampers
 367 with uniform slip load distributions. This is especially evident for medium to high-rise
 368 buildings. As illustrated in Figure 12, in some cases, using a fixed-wall system can lead to
 369 lower inter-story drift and roof displacement demands compared to the frames with friction-
 370 based wall dampers. However, fixed-wall systems considerably increase the total base shear
 371 and also transfer excessive additional axial loads to the columns and foundation (Figure 12 c
 372 and d). To ensure that these added axial force demands are within the load bearing capacity
 373 of the columns, the moment-axial load interaction curves of the column sections are

374 investigated. The example in Figure 13 shows that the critical moment-axial load
 375 combinations (at the first story) in the 10 and 15-story frames with fixed walls are generally
 376 beyond the load bearing capacity of the sections under the set of five synthetic spectrum-
 377 compatible earthquakes, while the friction wall dampers designed with the proposed
 378 methodology lead to acceptable design solutions. It can also be noted that fixed wall systems
 379 under seismic load will produce large tensile forces in the columns that can significantly
 380 reduce their moment resistance capacity.

381

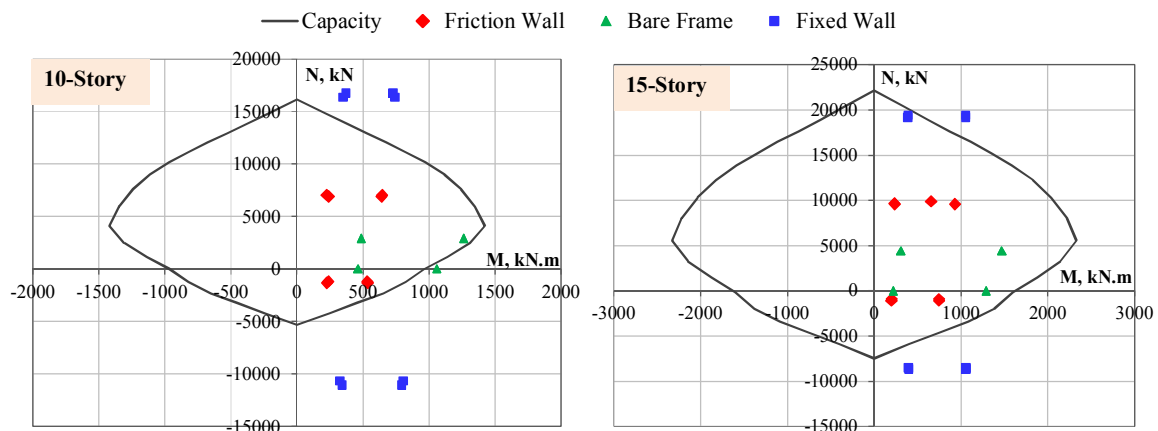


382

383

384 **Figure 12.** The ratio of (a) maximum drift; (b) maximum roof displacement; (c) maximum column
 385 axial load; (d) maximum base shear to the corresponding bare frames, average of five synthetic
 386 earthquakes

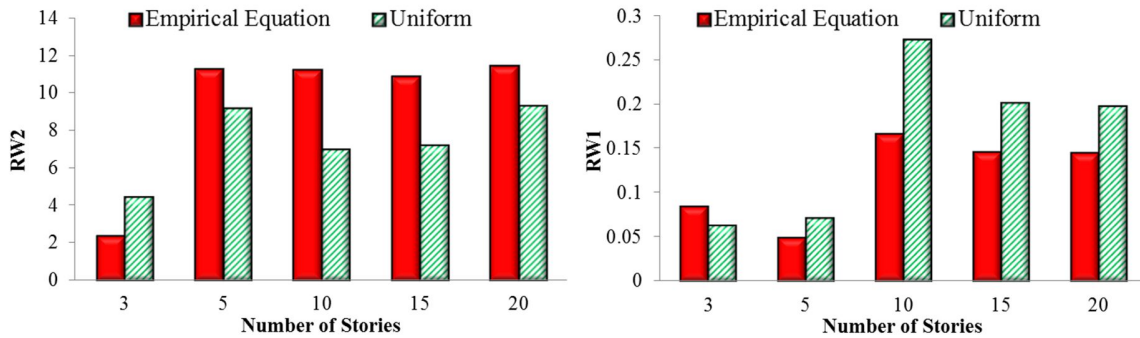
387



388

389 **Figure 13.** Comparison of the 1st floor column axial load-moment interaction for the bare frames and
 390 the frames designed with the empirical equation and fixed wall, average of five synthetic earthquakes

391 In the case of friction walls designed using the proposed empirical equation the moment-
 392 axial load demands on the columns are all within the acceptable range. This is a result of the
 393 limits to the story shear introduced by the friction connections. The results also indicate that
 394 the performance of the columns of the frames with more efficient design of friction walls can
 395 be better than those in the bare frames. The reason is that the increase in axial load of the
 396 columns in these frames is accompanied by a decrease in the maximum bending moments
 397 due to reduction of inter-story drifts. Figure 14 shows that the proposed slip load distributions
 398 in this study can lead to up to 61% higher energy dissipation capacity in the friction devices
 399 (i.e. higher R_{w2} factor) and up to 40 % lower energy dissipation demand in the structural
 400 elements (i.e. lower R_{w1} factor) compared to the conventional solutions.



401
 402 **Figure 14.** Energy dissipation parameters R_{w1} and R_{w2} as a function of number of stories, average of
 403 five synthetic earthquakes

404 GLOBAL DAMAGE INDEX

405 A linear cumulative damage model is used to calculate the overall damage index of the
 406 structure during seismic excitations by taking into account the changes in the energy
 407 dissipation capacity of the structure as a function of displacement demands (Miner, 1945;
 408 Teran-Gilmore and Jirsa, 2004). In this model it is assumed that the damages caused by
 409 plastic excursions are independent, while excursions are identified by using the Rainbow
 410 Counting Method suggested by Powell and Allahabadi (1987). In this study the inter-story
 411 inelastic deformation is chosen as the basic damage quantity, and the cumulative damage
 412 index after N excursions of plastic deformation is calculated using the following equation:

$$413 \quad DI_i = \sum_{j=1}^N \left(\frac{\delta_{pj}}{\delta_y} \right)^C \quad (6)$$

414 where (DI_i) is the cumulative damage index at i^{th} story, ranging from 0 for undamaged to
 415 1 for severely damaged stories, N is the total number of plastic excursions, δ_{pj} is the plastic

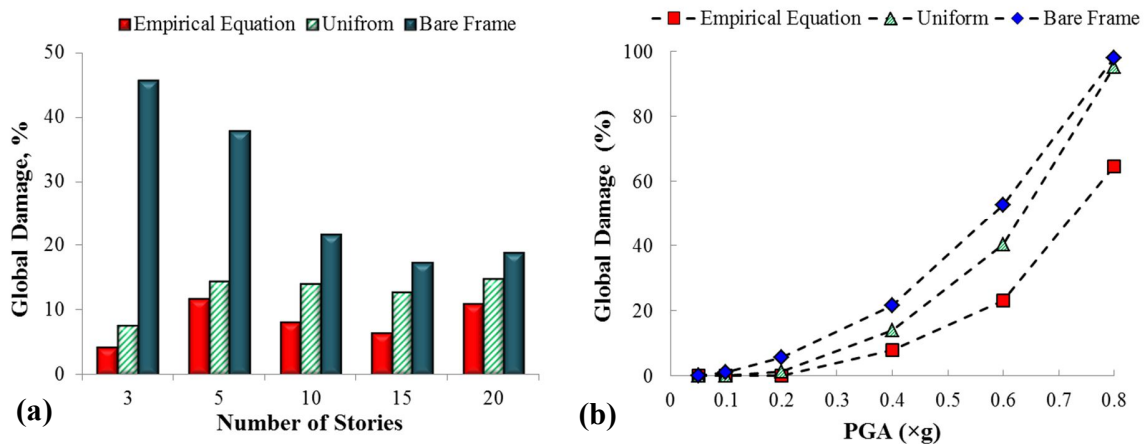
416 displacement of the j^{th} excursion, δ_y is the ultimate plastic displacement, and c is a structural
 417 parameter which accounts for the stability of the hysteretic behavior. In this study, c is
 418 considered to be 1.5, as suggested by Cosenza and Manfredi (1996) for damage analysis of
 419 reinforced concrete structures.

420 The global damage index (DI_g) evaluates the damage of the whole structure by
 421 considering the weighted average of the story damage indices. The following equation is used
 422 to calculate the global damage index of the structures:

$$423 \quad DI_g = \frac{\sum_{i=1}^n DI_i W_{pi}}{\sum_{i=1}^n W_{pi}} \quad (7)$$

424 where n is the number of stories, W_{pi} and DI_i are the dissipated energy and the damage
 425 index of the i^{th} story, respectively.

426 In Figure 15, the global damage indices of the bare frames under the set of five synthetic
 427 spectrum compatible earthquakes are compared with the frames with friction-based wall
 428 dampers designed using the proposed equation (Equation 5) and the uniform slip load
 429 distribution. In general the results indicate that friction-based dampers could significantly
 430 improve the seismic performance of the bare frames, especially for low to medium-rise
 431 buildings where the global damage index was reduced by up to 91%. Figure 15 (a) shows that
 432 friction dampers designed with the proposed equation could reduce the global damage index
 433 of the 3, 5, 10, 15 and 20-story frames by 45%, 19%, 43%, 50% and 26%, respectively,
 434 compared to conventionally designed dampers.



435
 436 **Figure 15.** Global damage index of (a) the bare frames compared to the frames with friction-based
 437 wall dampers designed using the proposed equation and uniform distribution and (b) the 10-story
 438 frame under different earthquake PGA scale factor, average of five synthetic earthquakes

439 The efficiency of the proposed optimization method is also investigated for different
440 earthquake intensity levels. Figure 15 (b) compares the global damage index (DI_g) of the 10-
441 story bare frame with the frames with friction wall dampers designed using Equation 5 and
442 uniform slip load distributions subjected to the set of five synthetic earthquakes with PGA
443 levels ranging from 0.05 to 0.8 g. It is shown that on average the friction wall dampers with
444 the slip load distribution suggested in this study always exhibit less global damage compared
445 to the frames with conventional friction walls at all PGA levels. The results in Figure 15 (b)
446 imply that the effectiveness of the wall dampers with a uniform slip load distribution was
447 considerably reduced at higher earthquake intensity levels (e.g. $PGA > 0.6$ g). This is because
448 using equal slip loads at all story levels led to a non-uniform distribution of lateral
449 displacement demands and consequently high local damage concentrated at some of the
450 stories (i.e. soft story failure), while the proposed slip load distribution resulted in a more
451 uniform distribution of story damage.

452 Although in general the seismic performance of friction wall dampers depends on the
453 frequency content of the input earthquake, number of stories and the earthquake intensity, the
454 outcomes of this study demonstrate that the dampers designed with the proposed method
455 consistently outperform those designed with uniform distribution of slip forces.

456 SUMMARY AND CONCLUSIONS

457 In this study, the efficiency of a friction-based wall system was investigated by extensive
458 nonlinear dynamic analyses on 3, 5, 10, 15, and 20-story RC frames subjected to six real and
459 a set of five synthetic design spectrum-compatible earthquakes. To obtain the most efficient
460 height-wise slip load distribution, five different distribution patterns were investigated,
461 including uniform, uniform cumulative, triangular cumulative, inverted triangular cumulative
462 and a distribution proportional to the story shear strengths. Based on the results presented in
463 this paper, the following conclusions can be drawn:

464 Uniform cumulative slip load distribution is usually the most effective pattern in terms of
465 increasing the energy dissipation capacity of the friction-based wall dampers. However,
466 irrespective to the slip load distributions, there is always an optimum range for the slip load
467 ratios (normalized to the story shear strength) that leads to minimum displacement demands
468 under design compatible earthquakes. For slip load ratios lower than the optimum value, the
469 effectiveness of the dampers can be limited due to the small energy dissipation in the friction

470 devices. Larger slip force ratios, however, may lead to connection lock-ups resulting in a
471 linear elastic response with large dynamic magnification and low energy dissipation. The
472 results show that the optimum range of the slip loads exponentially decreases with the
473 increase of the number of stories.

474 Based on the results of this study, an empirical equation was proposed to calculate a more
475 efficient slip load distribution for seismic strengthening/design of RC structures with
476 different number of stories. The friction wall systems designed based on the proposed
477 equation was shown to result in lower displacement demands (by up to 30%) and higher
478 energy dissipation capacities (by up to 61%), compared to the conventional systems with a
479 uniform slip load distribution.

480 It was shown that friction wall dampers designed with the proposed equation can
481 significantly reduce the displacement demands of the bare frames without large increase in
482 base shear. Although friction wall dampers impose additional axial loads to the adjacent
483 columns, it was shown that by using the proposed design method the axial loads generally
484 remain within the capacity of the column sections. However, if fixed panels are added to the
485 bare frame (as a retrofit measure) the maximum axial loads can be well beyond the maximum
486 capacity of the columns.

487 The results of nonlinear incremental dynamic analyses show that the friction dampers
488 designed with the proposed empirical equation can reduce the global damage index of the RC
489 frames with conventionally designed dampers by up to 43%. While the efficiency of the wall
490 dampers with a uniform slip load distribution was considerably reduced at higher earthquake
491 intensity levels, the suggested design solutions were efficient at all PGA levels.

492

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