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# 1 **Large-scale experimental investigation of a low-** 2 **cost PVC ‘sand-wich’ (PVC-s) seismic isolation** 3 **for developing countries**

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6 **and Nicholas A. Alexander,<sup>a)</sup>**

7         This study presents a large-scale experimental investigation on the seismic  
8         performance of an innovative, low-cost seismic isolation system for developing  
9         countries. It is based on the beneficial effect of the encapsulation of sand grains  
10        between two PVC surfaces on the initiation of sliding and the dissipation of seismic  
11        energy between the surfaces. A three-times scaled down, idealized, seismically  
12        isolated model of a prototype single-story structure located in Nepal is subjected to  
13        an ensemble of recorded earthquake ground motion excitations. The experimentally  
14        derived response of the seismically isolated structure is compared with the response  
15        of the corresponding fixed-base structure. This system is part of a wider hybrid  
16        design approach where the structure is designed to resist the seismic forces at the  
17        design acceleration level. The seismic isolation system sets an upper bound to the  
18        response of the structure for ground motion excitations exceeding the design level.

## 19 **INTRODUCTION**

20        The determination of economical and resource-effective engineering solutions for the  
21        protection of our communities from seismic hazard is of paramount importance to the safety  
22        and the sustainable growth of our communities. A wide spectrum of highly engineered seismic  
23        isolation techniques that focus on the modification of the response of buildings and bridges to  
24        earthquake ground motion excitation has been developed during the last four decades to  
25        increase the seismic safety of these structures (Kelly 1990, Buckle and Mayes 1990,  
26        Constantinou et al. 1992, Heaton et al. 1995, Naeim and Kelly 1999, Naeim 2019). The  
27        fundamental parameters that influence the frictional properties of the existing highly

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28 engineered sliding seismic isolation techniques have been quantified by Kumar et al. (2015),  
29 Furinghetti et al. (2019a) and Quaglioni et al. (2019).

30 Notwithstanding the significant contribution of these seismic isolation systems to the  
31 reduction of earthquake-induced structural damage and losses, the unambiguously high  
32 installation cost and the resource-demanding construction process inhibit their application to  
33 countries with limited material and financial resources (Kelly 2002, Yang et al. 2010).

34 In view of these limitations, several researchers have investigated the sliding and rolling  
35 frictional resistance of a broad variety of low-shear-strength material interfaces to facilitate the  
36 design of low-cost seismic isolation strategies based on the premise of a sliding energy  
37 dissipation mechanism. In the beginning of the 20<sup>th</sup> century, Calanteris (1909) proposed the  
38 design of a building founded on a layer of talc as a low-cost sliding seismic isolation system.  
39 A similar seismic isolation system, consisting of a layer of soil and a layer of soft mud was  
40 adopted by Wright (1977), who engineered the seismically isolated Imperial hotel in Tokyo.  
41 The application of low-cost sliding isolation systems to masonry structures was experimentally  
42 investigated by Li et al. (1989) and Nanda et al. (2016), who performed shaking table tests on  
43 the sliding response of a masonry structure based on different sliding interfaces. Jampole et al.  
44 (2016) conducted full-scale shaking table tests towards the implementation of a sliding seismic  
45 isolation system in low-rise lightweight residential buildings. The seismic behavior of  
46 lightweight structures seismically isolated with a system consisting of high-strength concrete  
47 slabs and a deformable steel rolling ball was investigated by Cilsalar and Constantinou (2019).

48 However, the challenge of the seismic protection of structures in developing countries for a  
49 wide variety of earthquake motion intensities remains a complex task that necessitates the  
50 consideration of a combination of seismic mitigation mechanisms, each being activated at a  
51 different ground motion acceleration level. Following the above line of thought, this study  
52 focuses on a hybrid design approach, combining two seismic mitigation mechanisms: The  
53 structure withstands seismic forces as normal up to the design acceleration and once this is  
54 exceeded, a sliding seismic isolation system acts as a fuse which effectively sets an upper  
55 bound to the forces that can be transferred to the building. This hybrid design approach was  
56 proposed by Tsiavos et al. (2019) investigating experimentally at small-scale the static and  
57 dynamic sliding response of a structure founded on a sand-rubber layer as a seismic isolation  
58 system for developing countries. The large-scale investigation of this seismic isolation system  
59 has led to significant rocking response, before the sliding response of the structure was  
60 triggered. Within this context, the determination of a seismic isolation system that facilitates

61 the activation of sliding at a relatively low acceleration threshold is critical for the economical  
62 design of a structure and the efficiency of the proposed hybrid design approach.

63 The mechanics of rolling-sliding behavior of structures can emerge as a natural means of  
64 decreasing the abovementioned sliding acceleration threshold, whose potential has still not  
65 been clearly established.

66 The quantification of the fundamental laws that govern the sliding and rolling frictional  
67 resistance between two contact interfaces dates back to several centuries ago. Coulomb (1785)  
68 stated that the friction of a rolling wheel is analogous to the vertical force acting on the wheel,  
69 and inversely proportional to the radius of the wheel. Dupuit (1842) incorporated the material  
70 deformability in the determination of the rolling friction by suggesting that the material  
71 subjected to a rolling motion of an object exhibits plastic deformation after rolling occurs.  
72 Bowden and Tabor (1956) attributed the sliding frictional resistance between two solid surfaces  
73 to two types of forces: first, an adhesion force that is inextricably linked to the shear strength  
74 of the interface between the two surfaces and the contact area between microscopically rough  
75 surface irregularities, defined as asperities. Second, a plastic deformation (plowing) force  
76 highly correlated with the relative hardness of the materials in contact and the uniformity of  
77 the grains of these materials (Shooter and Tabor 1952).

78 The aforementioned plowing component of friction can be significantly decreased by the  
79 encapsulation of sand particles between two hard surfaces in a ‘sand-wich’ configuration.  
80 O’Rourke et al. (1990) showed that the increase of the hardness of a polymer surface reduces  
81 substantially the friction of its interface against sand particles and suggested the manifestation  
82 of a rolling behavior accounting for the friction of sand particles against smooth polymer  
83 interfaces. Dove and Frost (1999) concluded that the interface of spherical, uniform particles  
84 against hard polymer (PolyVinyl Chloride-PVC) surfaces exhibits initiation of sliding at a  
85 significantly lower friction level compared to the case of angular, non-uniform particles.  
86 Ebrahimian et al. (2019) presented a numerical investigation on the interface shearing behavior  
87 of a thin granular soil layer, encapsulated by two rigid surfaces of different roughness.

88 The sliding seismic isolation system proposed in this study capitalizes on the presented low  
89 frictional resistance of sand particles against polymers. The PVC-sand-wich (PVC-s) seismic  
90 isolation is based on the inclusion (sandwiching) of a thin film of sand grains between two  
91 PVC sheets. This seismic isolation strategy belongs to the family of geotechnical seismic  
92 isolation (GSI) strategies, as defined by Tsang and Pitilakis (2019). The large-scale shaking  
93 table tests presented in this study show that the seismic isolation of a low-rise structure based

94 on this novel PVC-s sliding isolation system leads to an attractively low sliding acceleration  
95 threshold in the range of 0.25-0.3g.

96 Along these lines, the main goal of this study is to investigate means of low-cost  
97 enhancement of structural performance and in particular a hybrid design, which includes the  
98 seismic resistance of the structure at the design level and the activation of the PVC-s seismic  
99 isolation for the above-design earthquake intensities. In a realistic case the friction coefficient  
100 and several other parameters of the proposed seismic isolation system cannot be controlled but  
101 this is not a major issue given that PVC-s can only have a beneficial effect on the seismic  
102 response of the structure. The application of the presented hybrid design approach to masonry  
103 structures can be combined with standard strengthening of masonry walls using wire mesh as  
104 well as with the installation of a light roof on the structure that reduces its total mass.

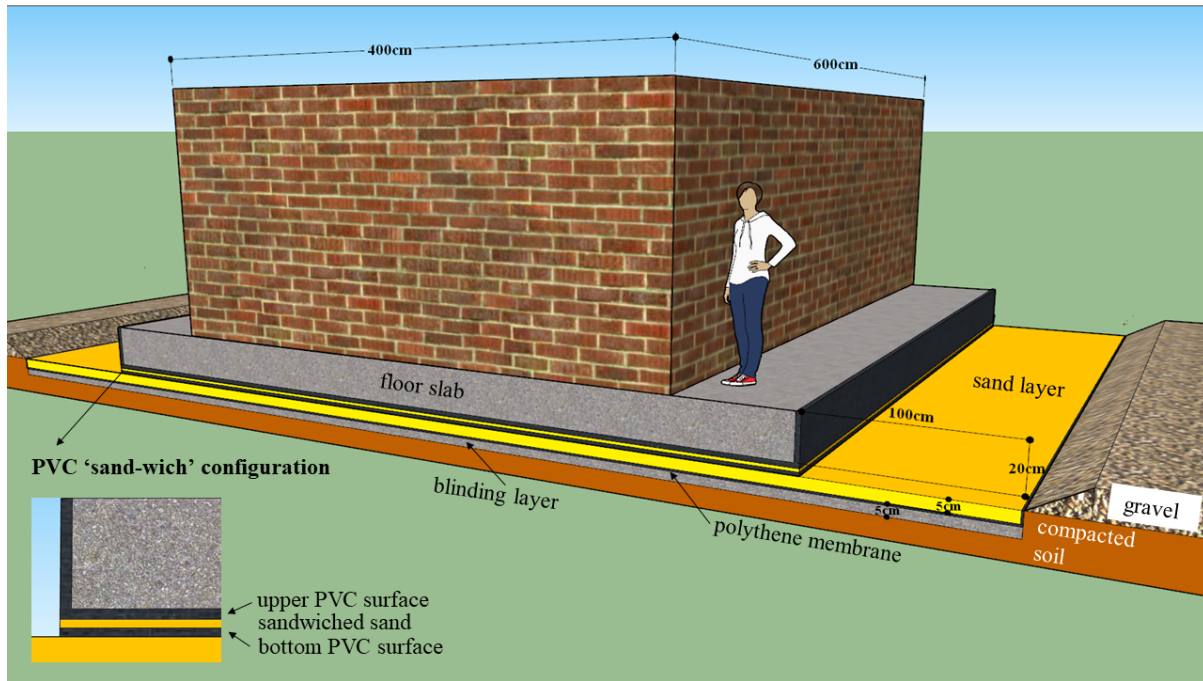
105 The PVC-s is the seismic isolation system that has been experimentally qualified in this  
106 study as a key component that facilitates the implementation of the presented hybrid design  
107 approach due to its attractive frictional characteristics and the initiation of sliding at a low  
108 acceleration threshold close to 0.25-0.3g. The comparison of the experimentally observed  
109 response of the seismically isolated structure to an ensemble of ground motion excitations with  
110 the experimentally derived fixed-base response illuminates the engineering merit of this  
111 seismic isolation system towards the reduction of the seismic forces and the seismic damage  
112 of low-rise structures in developing countries.

### 113 **ISOLATION CONCEPT FOR THE PROTOTYPE STRUCTURE**

114 The prototype is a one-storey unreinforced masonry structure with a light steel roof  
115 supported by 23cm thick masonry walls. The hybrid design approach presented in this study  
116 shares the same light roof concept with the prototype structure as a means of increasing the  
117 seismic safety of the structure. This structure is a typical example of a masonry one-classroom  
118 school building located in Nepal. The light-roof concept is commonly met in Nepal after the  
119 destructive 2015 Gorkha earthquake in the construction of structures known as Temporary  
120 Learning Centers (TLCs), providing safe educational spaces for children. The dimensions of  
121 the school building are shown in Fig. 1. The vertical stress on the foundation due to the weight  
122 of the structure is  $\sigma'_v=9\text{kPa}$ .

123 Fig. 1 elucidates the design of the proposed innovative PVC-s seismic isolation system. The  
124 fundamental design configuration entails the encapsulation of sand grains between two hard  
125 6mm thick PVC surfaces, positioned below the 20cm thick concrete foundation slab of the

126 structure. The encapsulation of sand grains of a predetermined mass facilitates a sliding  
127 behavior of the top PVC sheet against the bottom PVC sheet, enhanced by the low rolling  
128 resistance of the sand particles.



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**Figure 1.** Masonry school prototype structure located in Nepal (Dimensions in cm).

132 The application of the PVC-s seismic isolation to the prototype structure entails the  
133 following steps: A 10cm deep excavation is performed first at the site of the structure. The  
134 lateral extent of the excavation is wider than that of the intended foundation slab by 100cm,  
135 thus minimizing the risk of pounding of the foundation slab against the surrounding soil due to  
136 an excessive sliding displacement of the structure. A 30° inclined, 10cm thick gravel layer  
137 based on the surrounding soil is used as a restraining mechanism to ameliorate the  
138 consequences due to a potential exceedance of the maximum expected sliding displacement  
139 estimates.

140 A 5cm thick sand layer is enclosed at its bottom and lateral sides by a polythene membrane  
141 sheet and based on a 5cm thick sub-foundation blinding layer, aimed at the formation of a  
142 levelled, stiff base below the seismically isolated structural system. The unreinforced blinding  
143 layer is founded on the existing soil at the site of the structure below the 10cm deep excavation  
144 level, which should be compacted before the concrete casting of the blinding layer.

145 The bottom PVC surface of the proposed isolation system is placed on the 5cm thick sand  
146 layer. A thin film of sand particles is placed above the bottom PVC layer and below the upper

147 PVC layer, thus creating the intended ‘sand-wich’ configuration (Fig. 1). The 20cm thick  
148 concrete slab is casted on the upper PVC surface, which remains positioned below the  
149 foundation slab as an undegradable, permanent formwork. The masonry structure is  
150 constructed on the concrete slab, after it hardens.

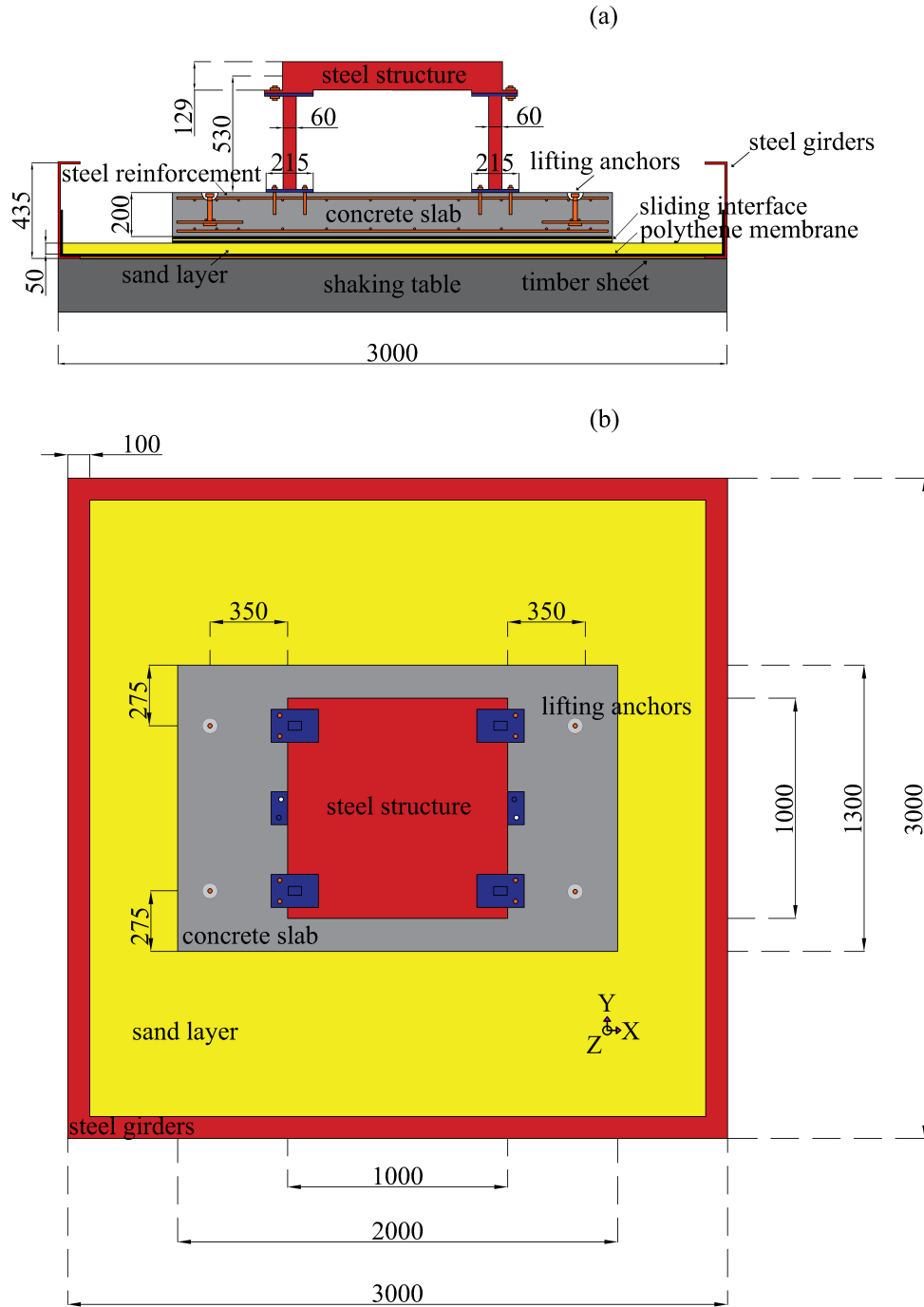
151 The low cost of the presented PVC-s seismic isolation comparing to the existing highly  
152 engineered isolation systems emerges from the minimization of the construction and  
153 transportation cost of the isolation system through its simple construction based on materials  
154 that are locally available in developing countries, such as PVC and sand. Furthermore, the  
155 presented floor slab-blinding layer configuration leads to a substantially lower construction  
156 cost compared to the existing highly engineered seismic isolation systems, requiring the  
157 construction of two reinforced concrete slabs: One slab creating a diaphragm above the  
158 isolation devices and one foundation slab below the level of these devices. However, the slab  
159 configuration of the proposed low-cost isolation system shown in Fig. 1 comprises the  
160 construction of only one reinforced concrete floor slab above the isolation system and a 5cm  
161 thick unreinforced blinding concrete layer below the isolation system. Moreover, the ease of  
162 the application of the presented seismic isolation, which does not necessitate a project specific  
163 testing of the isolation system, facilitates an additional cost reduction and higher applicability  
164 to a developing country compared to the existing seismic isolation systems.

165

## EXPERIMENTAL SETUP

166 A three-times-scaled down model of the aforementioned masonry prototype structure is  
167 designed as shown in Fig. 2. The test structure is made out of steel due to the low vulnerability  
168 and the transportability of the structure convenient for a parametric investigation and it is  
169 designed to meet three design objectives: First, the height of the steel model is chosen to  
170 correspond to 2/3 of the height of the scaled-down height of the masonry walls of the prototype  
171 (83cm), thus maintaining the acting point of the equivalent seismic force between the scaled-  
172 down steel frame and its masonry counterpart. Second, the model structure is designed to have  
173 the same fixed-base vibration period  $T_x=0.085s$  (frequency  $f_x=12Hz$ ) in X-Direction of shaking  
174 (Fig. 2) with the period of the prototype structure, calculated using the stiffness estimation for  
175 unreinforced masonry walls proposed by Wilding and Beyer (2018). Third, the sand layer  
176 below the model structure is subjected to the same vertical foundation stress level  $\sigma_v=9kPa$  as  
177 the prototype structure.

178 The investigated structure is a 1t steel structure and it is based on a 20cm thick, 1.3t  
 179 reinforced concrete slab. The concrete slab is founded on a 5cm thick sand layer, deposited  
 180 with zero-height drop. The density of the sand layer is  $1540\text{kg/m}^3$ . The abovementioned ratio  
 181 is defined in this study as sand surface density ( $\text{g/m}^2$ ). The properties of the Leighton Buzzard  
 182 14-25 sand used in this study are summarized in Table 1.



183 **Figure 2.** (a) Cross-section and (b) plan view of the design of the experimental setup (Dimensions in  
 184 mm).  
 185



186 **Table 1.** Characteristics of the Leighton Buzzard 14-25 sand used in this study.

Specific gravity $G_s$	Void ratio $e_{max}$	Void ratio $e_{min}$	Mean size $D_{50}$ (mm)	$C_u=D_{60}/D_{10}$	$C_g=D_{30}^2/D_{60}D_{10}$
2.65	0.84	0.53	0.883	1.439	0.996

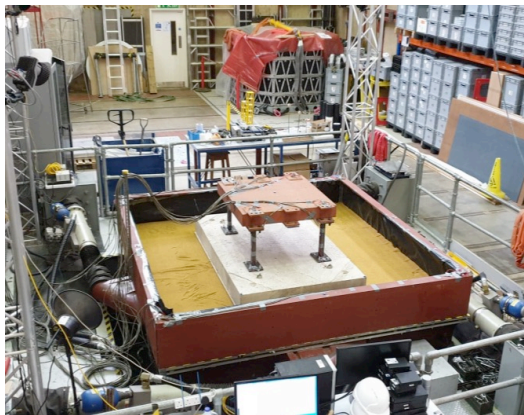
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188 The seismic response of this structure is assessed based on experimental large-scale testing  
 189 at the 3mx3m shaking table of University of Bristol. The shaking table has a six-degree-of-  
 190 freedom motion testing capability. The maximum payload of the table is 15t. Depending on  
 191 the loading, the shaking table can apply acceleration amplitudes up to 5g.

192 Four C-shaped steel girders are fixed on the edges of the shaking table of University of  
 193 Bristol to enclose the sand layer and prevent the structure from excessive sliding  
 194 displacements. A polythene membrane is placed around and below the sand layer to inhibit any  
 195 leakage of sand grains. A timber sheet is attached above the shaking table and below the  
 196 polythene membrane to act as a secondary protection mechanism from leakage of sand in case  
 197 of damage in the polythene membrane.

198 **INSTRUMENTATION**

199 The constructed experimental setup and the applied instrumentation are shown in Fig. 3.  
 200 The instrumentation of the experimental setup shown in Fig. 4 consists of 25 accelerometers  
 201 and 16 infrared displacement markers, which are tracked by 5 infrared high-speed cameras.  
 202 The cameras are fixed on a rigid aluminium frame, 3m above the shaking table. Each  
 203 accelerometer (red colour) measures the acceleration in one direction, while each marker  
 204 (yellow colour) tracks the displacement in three directions at selected locations of interest. All  
 205 measurements are synchronized.



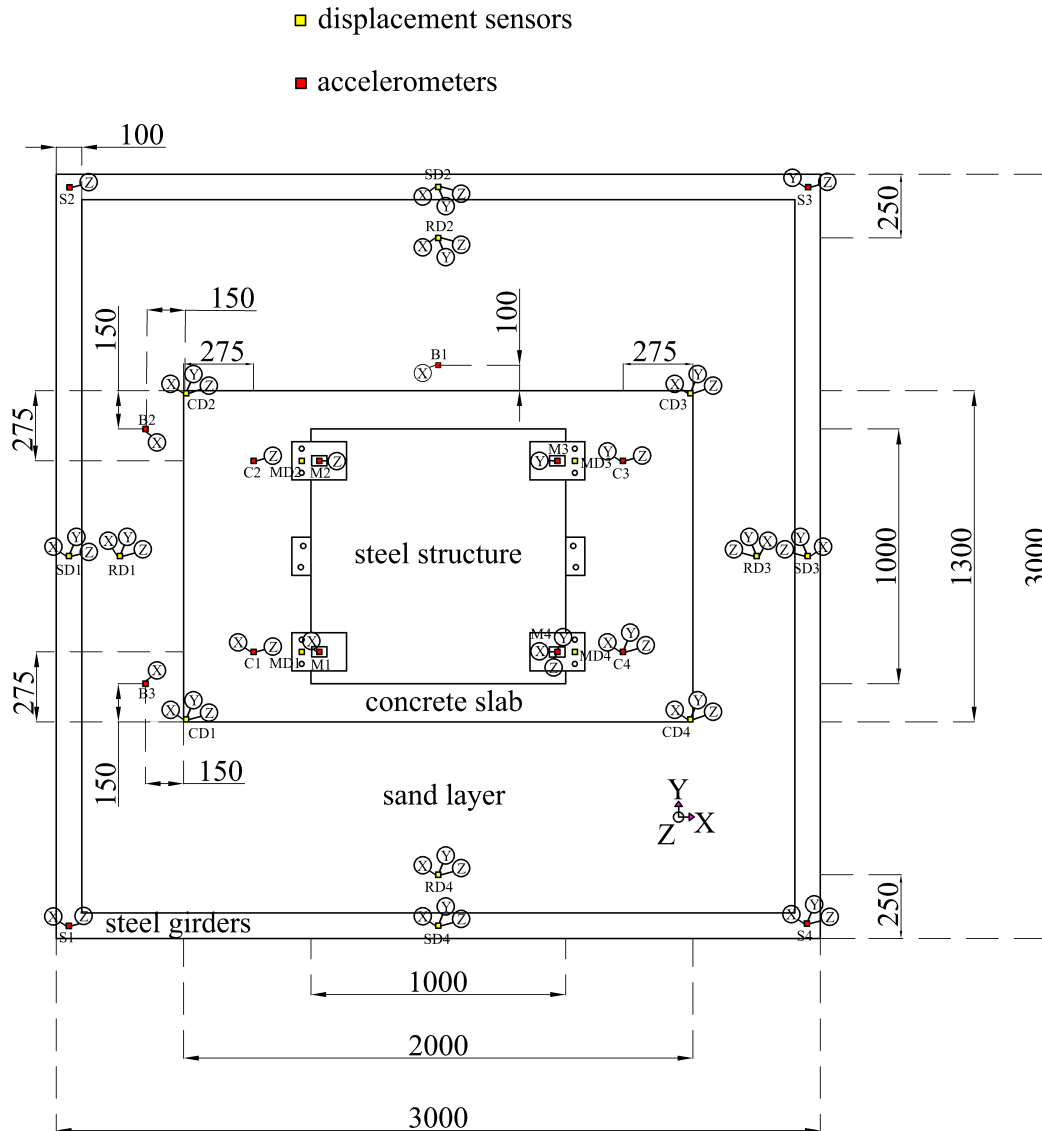
206 (a)



207 (b)

206

207 **Figure 3.** (a) Overview and (b) side view of the constructed experimental setup.



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**Figure 4.** Instrumentation plan of the experimental setup (Dimensions in mm).

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**INSTALLATION OF PVC SURFACES AND DEPOSITION OF SAND**

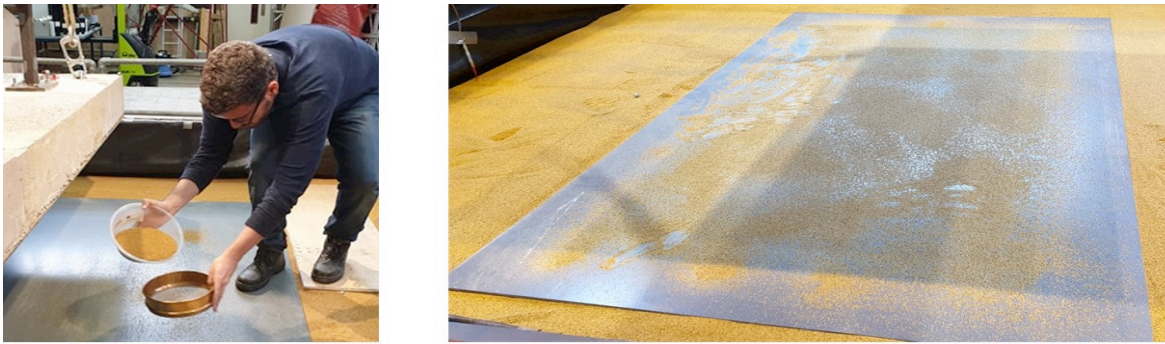
211

The PVC sliding surfaces were installed parallel to each other and the sand foundation layer, thus not considering possible inclinations, which could be unintentionally formed during the application of the proposed system in the field. However, Furinghetti et al. (2019b) suggest that the consideration of these inclinations leads to low variation of the fundamental response parameters (displacement, base shear).

216

The deposition of sand between the two PVC sheets is performed as shown in Fig. 5. The use of a 2.2mm sieve facilitated the uniform distribution of the sand along the bottom PVC surface, thus allowing a uniform encapsulation of the sandwiched sand between the two PVC sheets.

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(a)

(b)

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221

**Figure 5.** (a) Deposition of sandwiched sand and (b) bottom PVC surface after sand deposition.

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### TESTING PROTOCOL

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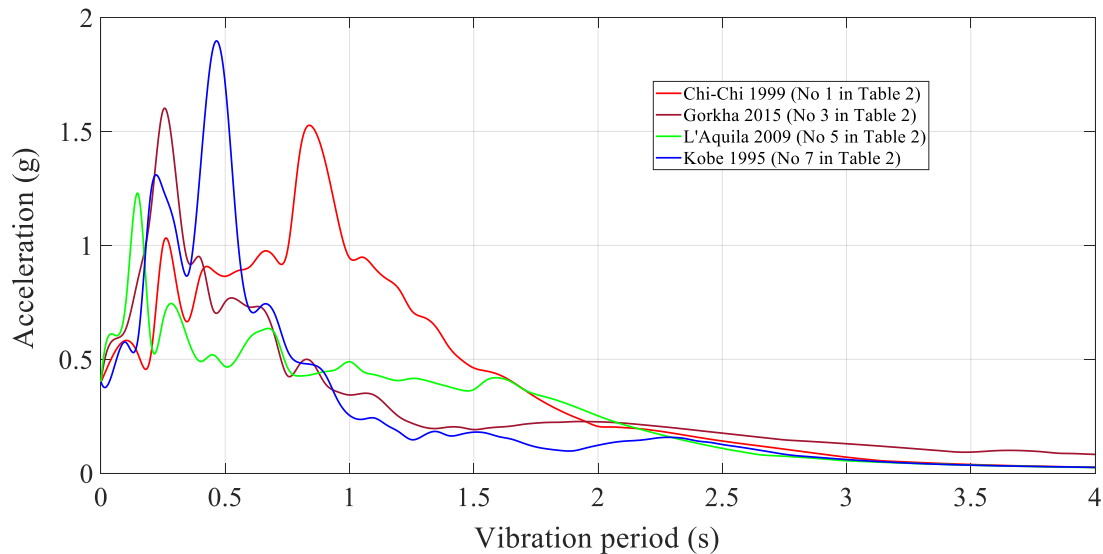
A white-noise test with amplitude  $A=0.05g$  and frequency range  $f=0-100Hz$  is performed for all experimental configurations presented in this study for the determination of the fundamental frequencies of the structural system. A spectrum analysis is employed to compute the Frequency Response Function (FRF) between the measured acceleration input and the measured acceleration output on the top of the steel slab of the structure (M1 in Fig. 4). The peaks of the FRF correspond to the modal frequencies. The experimentally derived, flexible-base fundamental vibration period of the presented structural system (Fig. 2-4) in X-Direction is  $\tilde{T}_x=0.09s$  ( $\tilde{f}_x=11Hz$ ). The small thickness of the bottom sand layer (5cm) is chosen to minimize the rocking response of the structure due to potential soil-structure-interaction effects. Accordingly, the measured period lengthening  $\tilde{T}_x/T_x=1.05$  due to soil-structure-interaction with respect to the fixed-base period value  $T_x$  lies in the low period-lengthening range of the values presented by Stewart and Fenves (1998), Mylonakis and Gazetas (2000), Taciroglu et al. (2016) and Star et al. (2019) for a wide range of structural systems.

The structure is subjected to nine strong earthquake ground motion excitations of different frequency content, obtained from the PEER NGA Database (2018) and the USGS Center for Engineering Strong Motion Data (2018). The horizontal components of the selected ground motion records 1-8 shown in Table 2 are scaled to two peak ground acceleration (PGA) levels: 0.4g and 0.6g and applied in X-Direction (Fig. 2, 4). The horizontal components of L'Aquila 2009 ground motion record (No. 9) are scaled to a peak ground acceleration (PGA) level of 0.5g. The three components (two horizontal and one vertical) of this record are applied simultaneously in directions X, Y and Z (Fig. 2, 4). The acceleration response spectra of the ground motion records considered in this study, scaled at a PGA=0.4g, are shown in Fig. 6.

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246

**Table 2.** Testing protocol of real earthquake records used in this study (PEER NGA Database, 2018 and USGS Center for Engineering Strong Motion Data, 2018).

No.	Date	Earthquake and Site	$M_w$	$R$ (km)	Component	Scaled PGA (g)
1	21/9/1999	Chi-Chi, CHY080	7.6	2.69	CHY080-E	0.4
2	21/9/1999	Chi-Chi, CHY080	7.6	2.69	CHY080-E	0.6
3	25/4/2015	Gorkha, KTP	7.8	75.8	KTP-NS	0.4
4	25/4/2015	Gorkha, KTP	7.8	75.8	KTP-NS	0.6
5	6/4/2009	L'Aquila, Parking	6.3	5.38	LAQ-AM043XTE	0.4
6	6/4/2009	L'Aquila, Parking	6.3	5.38	LAQ-AM043XTE	0.6
7	17/1/1995	Kobe, Nishi-Akashi	6.9	7.08	KOBE_NIS000	0.4
8	17/1/1995	Kobe, Nishi-Akashi	6.9	7.08	KOBE_NIS000	0.6
9	6/4/2009	L'Aquila, Parking	6.3	5.38	LAQ-AM043XTE LAQ-AM043YLN LAQ-AM043ZUP	0.5



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**Figure 6.** Acceleration response spectra of the ground motion records used in this study, scaled at a PGA=0.4g.

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### DIMENSIONAL ANALYSIS

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The maintenance of the similitude between the sliding behavior of the model structure and the prototype is fulfilled through the preservation of two dimensionless ratios  $\Pi_1$ ,  $\Pi_2$ . The dimensionless strength ratio  $\Pi_1 = \mu g / \text{PGA}$  ( $\mu$  being the static friction coefficient and  $g$  the acceleration of gravity) expresses the strength of the sliding interface relative to the ground motion intensity. The vibration period ratio  $\Pi_2 = \tilde{T}_x / T_g$  represents the relation between the flexible-base period  $\tilde{T}_x$  of the structural system over the predominant period of the excitation

257  $T_g$ , defined as the period where the 5% velocity spectrum attains its maximum (Miranda and  
258 Bertero 1994, Mylonakis and Gazetas 2000).

259 The friction coefficient  $\mu$  of an interface between a polymer surface and a granular material  
260 (Dove and Frost 1999) varies for different vertical stress levels. To avoid further complications,  
261 the sandwiched sand grains and the sand layer below the foundation of the model structure are  
262 subjected to the same vertical stress level  $\sigma'_v=9\text{kPa}$  as the foundation stress level below the  
263 prototype structure. Therefore, the friction coefficient and the associated dimensionless  
264 strength ratio are preserved between the model and the prototype structure, thus maintaining  
265 the similitude in the sliding behavior of the two structures. The model structure is designed to  
266 have the same elastic fixed-base vibration period  $T_x=0.085\text{s}$  with the prototype structure. The  
267 measured flexible-base vibration period value of the structure  $\tilde{T}_x$  is very close to the  
268 corresponding fixed-base value  $T_x$ . Thus, the frequency characteristics of the applied ground  
269 motions remain unchanged with respect to the original earthquake ground motion records and  
270 the ratio  $\tilde{T}_x/T_g$  is maintained between the model and the prototype structure.

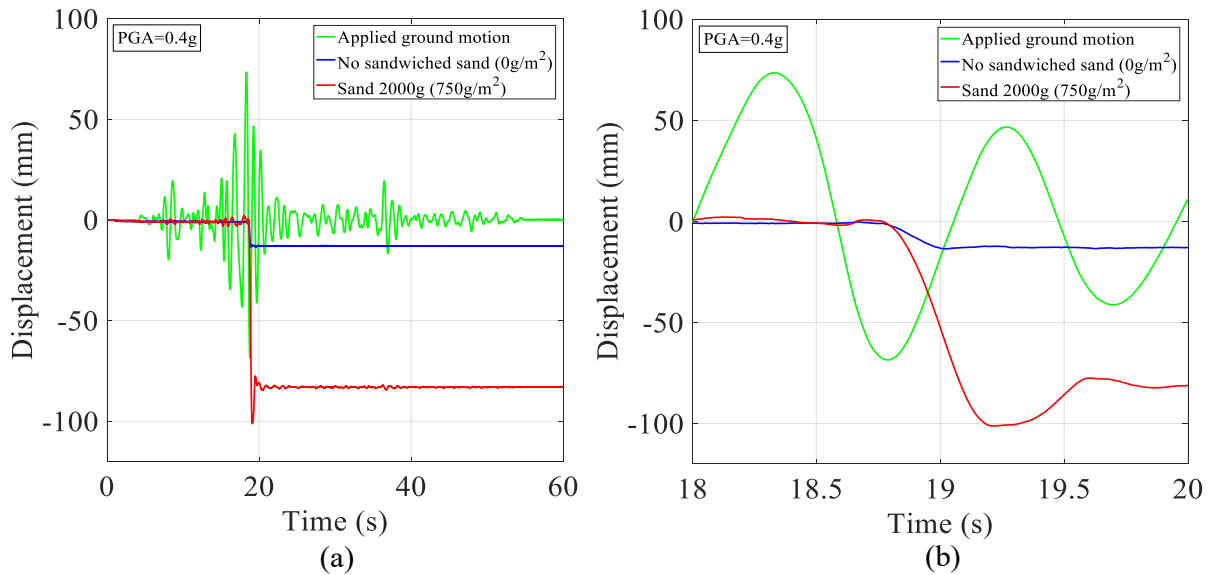
## 271 **EXPERIMENTAL INVESTIGATION**

### 272 **SLIDING BEHAVIOR OF PVC-S SEISMIC ISOLATION**

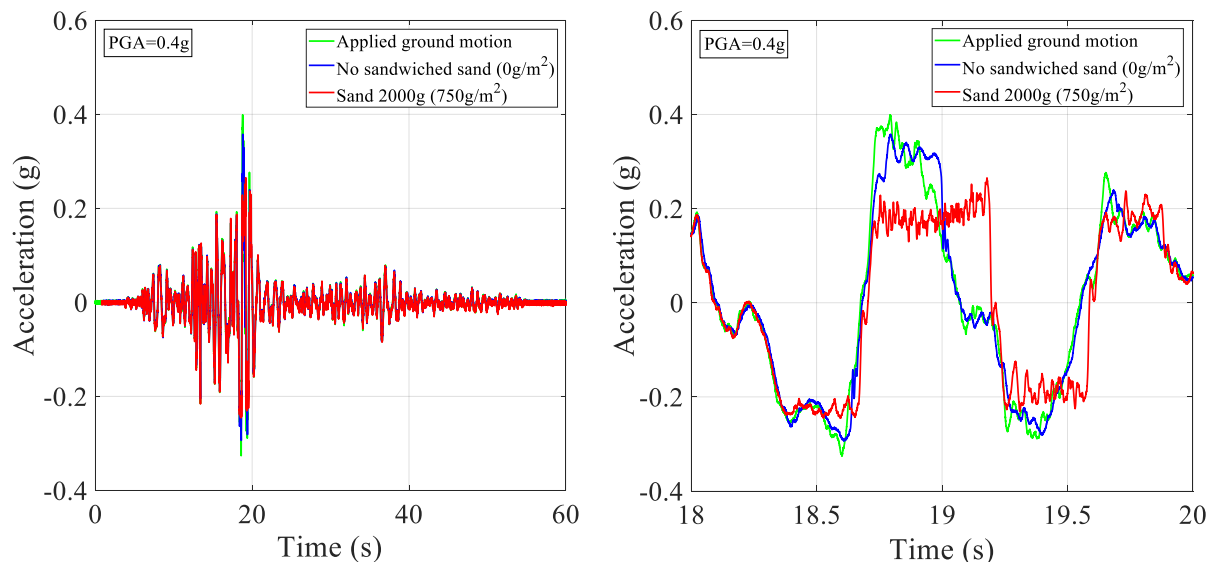
273 The hybrid design approach presented in this study has explored different seismic isolation  
274 systems as a means of seismic protection mechanisms, focusing on minimizing the ground  
275 motion acceleration threshold after which sliding occurs. The large-scale investigation of the  
276 seismic isolation of the structure presented in Fig. 2 using a sand-rubber layer (Tsiavos et al.  
277 2019) of thickness varying from 5cm-20cm has led to significant rocking response before the  
278 manifestation of the sliding behavior. Hence, the exploration of a seismic isolation system  
279 consisting of a granular layer with a minimum thickness of 5cm emerged as a necessity derived  
280 from the intended increase of vertical stiffness below the sliding interface of the structure. The  
281 choice of the PVC-s as a sliding seismic isolation system and the design configuration shown  
282 in Fig. 2 are an outcome of an iteratively determined balance between the attractive  
283 experimentally derived frictional properties of sand against polymers (O'Rourke et al. 1990,  
284 Dove and Frost 1999) and the desirable increase of vertical stiffness below the foundation slab  
285 of the structure.

286 The favorable role of the encapsulation of sand between the two PVC sheets and the  
287 sequence of the activation of the two inherent energy dissipation mechanisms, incorporated in

288 the design of the proposed seismic isolation system are illustrated in Fig. 7, 8: The presence of  
 289 uniformly distributed 2000g of sandwiched sand (surface density  $750\text{g/m}^2$  with respect to the  
 290 bottom PVC surface) triggers the initiation of sliding of the upper PVC sheet against the bottom  
 291 PVC sheet of the model structure subjected to Chi-Chi 1999 ground motion excitation (No. 1  
 292 in Table 2).



293  
 294 **Figure 7.** (a) Full and (b) magnified sliding displacement time history response of the concrete slab  
 295 relative to the shaking table motion (mean value of CDX-SDX in Fig. 4) subjected to Chi-Chi 1999  
 296 ground motion record (No. 1 in Table 2) for two different sand configurations between the two PVC  
 297 sheets.



298  
 299 **Figure 8.** (a) Full and (b) magnified acceleration time history response of the concrete slab (mean value  
 300 of C1X and C4X in Fig. 4) subjected to Chi-Chi 1999 ground motion record (No. 1 in Table 2) for two  
 301 different sand configurations between the two PVC sheets.

302 This sliding behavior expressed by a maximum displacement of 10cm (Fig. 7) is attributed  
303 to the exceedance of the rolling frictional resistance of the sand grains and leads to a maximum  
304 acceleration response of 0.25g (Fig. 8) at the level of the concrete slab (C1X, C4X in Fig. 4).  
305 This maximum acceleration response indicates the value of the friction coefficient of the  
306 interface  $\mu=0.25$  and is significantly lower than the corresponding acceleration response value  
307 of 0.35g, observed for the case in which the presented seismic isolation system does not include  
308 any sand between the two PVC sheets (Fig. 8). In this case, there is no relative displacement  
309 between the two PVC sheets, which exhibit a sliding behavior expressed by a maximum  
310 displacement of 1.3cm against the 5cm thick sand sub-foundation layer (Fig. 7).

311 The first sliding mechanism of the presented isolation system is activated between the two  
312 PVC sheets at a PGA=0.25g. The secondary sliding mechanism is activated, in case of no  
313 sandwiched sand enclosure, between the bottom PVC sheet and the sand sub-foundation layer  
314 at a PGA=0.35g. The existence of two sliding mechanisms illuminates the redundancy and the  
315 robustness of the presented dual seismic isolation system.

316 A parametric investigation of the acceleration response of the test structure subjected to Chi-  
317 Chi 1999 ground motion excitation (No. 1 in Table 2) for varying sand surface density values  
318 is performed. The experimentally derived results of this investigation are shown in Table 3 and  
319 Fig. 9. Two sliding events (first sliding event:  $18.7s \leq t \leq 19.2s$ , second sliding  
320 event:  $19.2s \leq t \leq 19.6s$ ) were used for the statistical evaluation of the friction coefficient  $\mu$  during  
321 the excitation of the structure. The mean, standard deviation and 95% confidence interval  
322 values of the friction coefficient  $\mu$  derived from the sliding events magnified in Fig. 9b are also  
323 shown on Table 3. Evidently, there is a range of sand surface densities between 40 and 750g/m<sup>2</sup>  
324 that lead to comparable performance in the response of the seismically isolated structure,  
325 expressed by the lowest mean friction coefficient value. Among these values, the sand surface  
326 density value of 750g/m<sup>2</sup> is recommended in this study as the one yielding the lowest dispersion  
327 of the friction coefficient value, leading to the lowest 95th percentile (Table 3). The  
328 aforementioned optimal range of sand surface density values determined for this ground  
329 motion excitation is generalized through the excitation of the structure by the ground motions  
330 presented in Table 2.

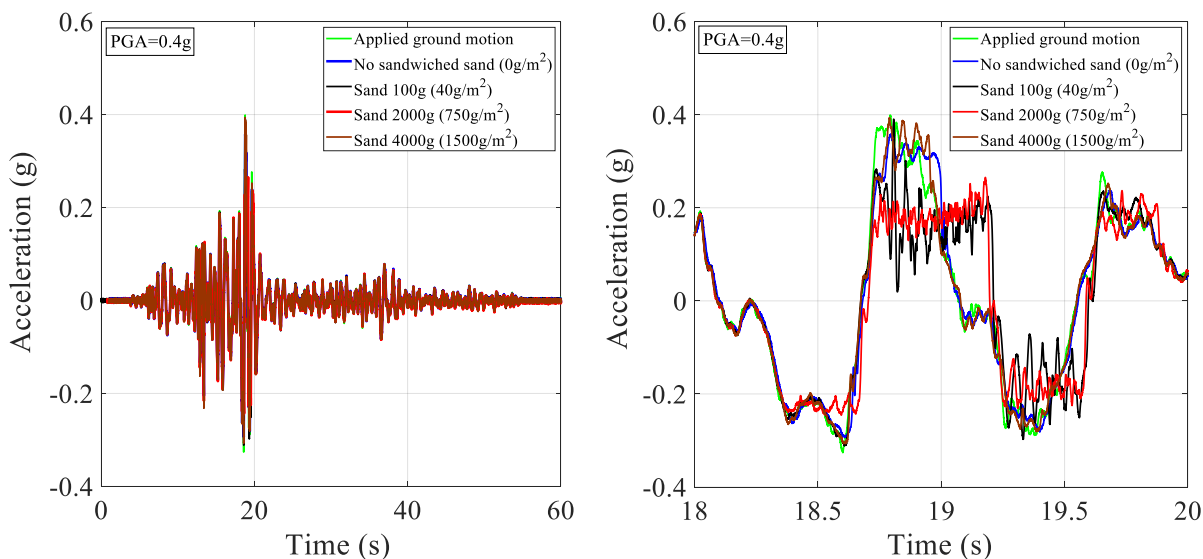
331 As shown in Fig. 9 an increase of the sand surface density from 100g (40g/m<sup>2</sup>) to 2000g  
332 (750g/m<sup>2</sup>) reduces the maximum acceleration response and the standard deviation of the  
333 oscillations observed during the sliding phase of the upper PVC layer against the bottom PVC

334 layer. This reduction in the maximum of the acceleration response is consistent with the  
 335 decrease of the corresponding 95th percentile value of the friction coefficient from 0.29 to  
 336 0.23 (Table 3), although the mean friction values for the two surface densities are comparable,  
 337 namely 0.179 and 0.187. Nevertheless, a further increase of this amount from 2000g to 4000g  
 338 (1500g/m<sup>2</sup>) is detrimental due to the increase of the maximum acceleration response and the  
 339 associated friction coefficient to a 95th percentile value  $\mu=0.34$  and a mean value of  $\mu=0.3$ .

340 **Table 3.** Experimentally derived mean, standard deviation and 95% confidence interval values of  
 341 friction coefficient  $\mu$  for varying sand surface density values.

PGA (g)	Sand surface density (g/m <sup>2</sup> )	Mean $\hat{\mu}$	Standard deviation $\sigma$	95% confidence interval ( $\hat{\mu} \pm 2\sigma$ )
0.4	0	0.285	0.015	$0.25 \leq \mu \leq 0.32$
0.4	40	0.179	0.056	$0.07 \leq \mu \leq 0.29$
0.4	750	0.187	0.024	$0.14 \leq \mu \leq 0.23$
0.4	1500	0.302	0.020	$0.26 \leq \mu \leq 0.34$

342



343

344 **Figure 9.** (a) Full and (b) magnified acceleration time history response of the concrete slab (mean value  
 345 of C1X and C4X in Fig. 4) subjected to Chi-Chi 1999 ground motion record (No. 1 in Table 2) for  
 346 varying amounts of sandwiched sand between the two PVC sheets.

347 The confirmation of the repeatability of the beneficial effect of the proposed seismic  
 348 isolation system is an essential step towards the application of this seismic isolation strategy to  
 349 structures in earthquake-prone areas. The investigation of the response of the structure for  
 350 subsequent ground motions simulates the effect of aftershocks or successive earthquake events  
 351 on the efficiency of the presented seismic isolation system when that is already activated and  
 352 displaced. The sliding displacement and acceleration response of the model structure to three

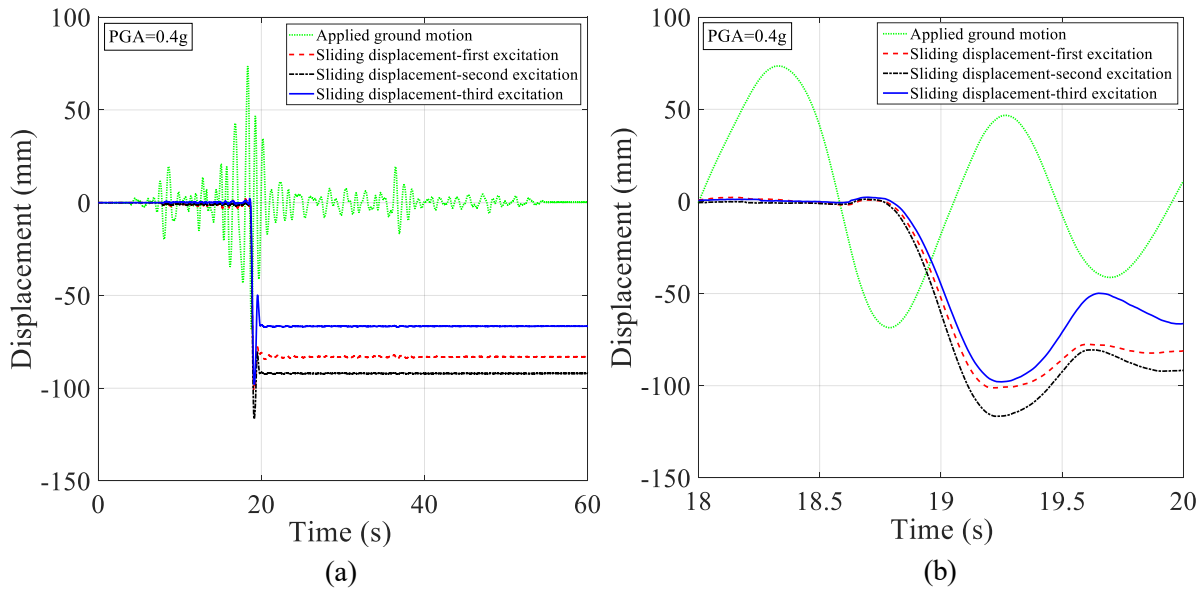


353 subsequent Chi-Chi 1999 ground motion excitations of  $PGA=0.4g$  is shown in Fig. 10, 11,  
354 respectively.

355 As shown in Fig. 10, the maximum and the residual components of the sliding displacement  
356 of the model structure (relative to the shaking table) due to the applied subsequent ground  
357 motion excitations deviate roughly 20% from the corresponding values attributed to the first  
358 applied ground motion excitation.

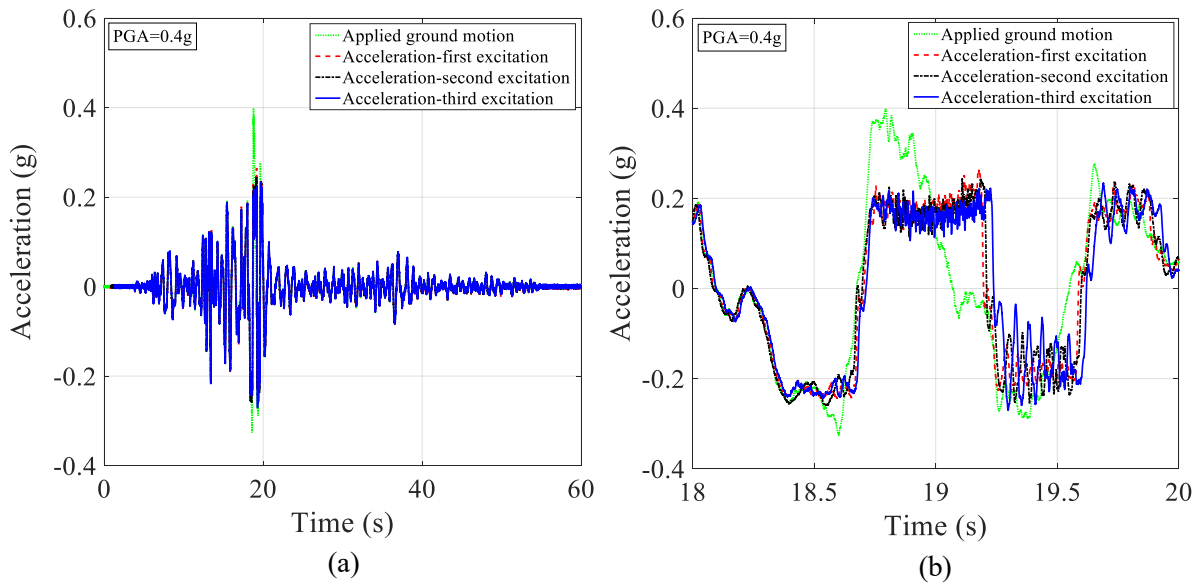
359 Similar to the system developed by Jampole et al. (2016), the proposed seismic isolation  
360 system does not provide any recentering force, but instead relies on friction alone. The residual  
361 displacement obtained for this ground motion excitation corresponds to the maximum residual  
362 displacement amplitude obtained in this study, reaching roughly 5% of the concrete slab width,  
363 namely 10cm. However, the presence of potential excessive sliding displacements is explicitly  
364 accounted in the design of the prototype structure by the wide extent of the excavation and the  
365 presence of additional space (margin) surrounding the foundation slab of the structure.  
366 Furthermore, the recentering of the presented one-storey prototype structure, displaced at  
367 maximum by 10cm from the original configuration after the earthquake event could be  
368 performed via a pulling force created by a truck. Hence, the risk of pounding due to these  
369 displacements is minimized. Future investigations could more precisely determine the required  
370 displacement safety margin for the prevention of this pounding. This determination should be  
371 based on an optimal balance between the cost required for the construction of this margin and  
372 the probability of pounding due to one or successive ground motions leading to a residual  
373 displacement of the structure on the same direction with the previous ground motion (Jampole  
374 et al. 2016).

375 The maxima of the acceleration response indicating the exceedance of the frictional  
376 resistance of the interface due to the subsequent excitations differ less than 10% from the values  
377 measured during the first excitation (Fig. 11). The presented stability of the frictional  
378 characteristics of the interface and the sliding behavior of the model structure for successive  
379 ground motion excitations confirm the repeatability of the beneficial effect of the proposed  
380 seismic isolation technique.



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**Figure 10.** (a) Full and (b) magnified sliding displacement time history response of the concrete slab relative to the shaking table motion (mean value of CDX-SDX in Fig. 4) for three repetitions of the same Chi-Chi 1999 ground motion record (No. 1 in Table 2).



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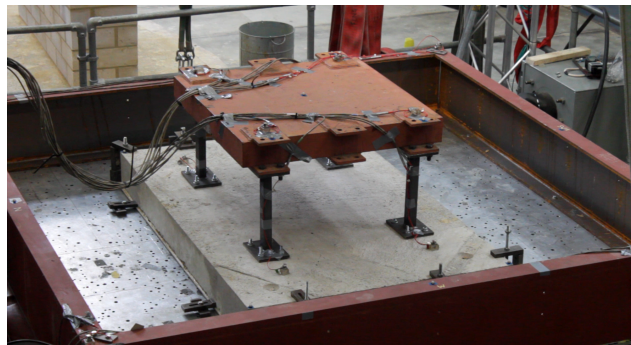
**Figure 11.** (a) Full and (b) magnified acceleration time history response of the concrete slab (mean value of C1X and C4X in Fig. 4) for three repetitions of the same Chi-Chi 1999 ground motion record (No. 1 in Table 2).

389 **SLIDING SYSTEM VERSUS FIXED-BASE STRUCTURE**

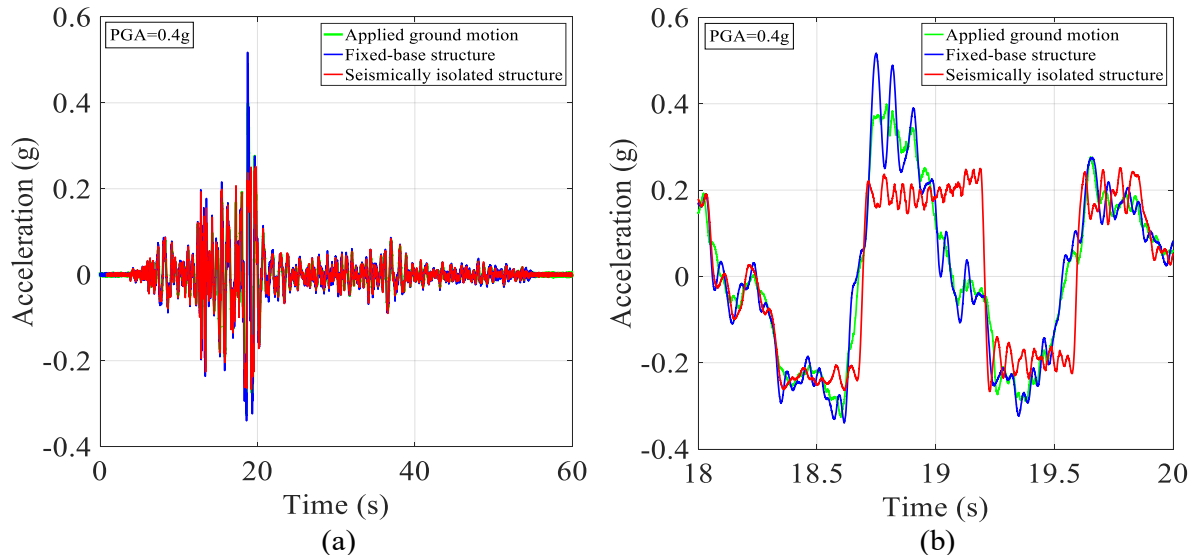
390 The favorable effect of the proposed seismic isolation system is further highlighted through  
391 the comparison of the acceleration response of the seismically isolated steel structure with the  
392 response of the same structure, after the rigid attachment of its base to the shaking table (Fig.

393 12). The experimentally derived vibration period of the fixed-base structural system  $T_x=0.075s$   
 394 ( $f_x=13Hz$ ) is in good agreement with the design value of  $T_x=0.085s$  ( $f_x=12Hz$ ).

395 As shown in Fig. 13, the experimentally derived maximum acceleration of the seismically  
 396 isolated structure subjected to Chi-Chi 1999 ground motion excitation (No. 1 in Table 2) is 2.5  
 397 times lower than the maximum acceleration of the fixed-base structure subjected to the same  
 398 ground motion excitation. The reduction of the maximum seismic acceleration and the  
 399 associated seismic force acting on the structure illustrates the efficiency of the proposed  
 400 seismic isolation technique on the mitigation of seismic damage.



401  
 402 **Figure 12.** Experimental setup of the fixed-base structure.

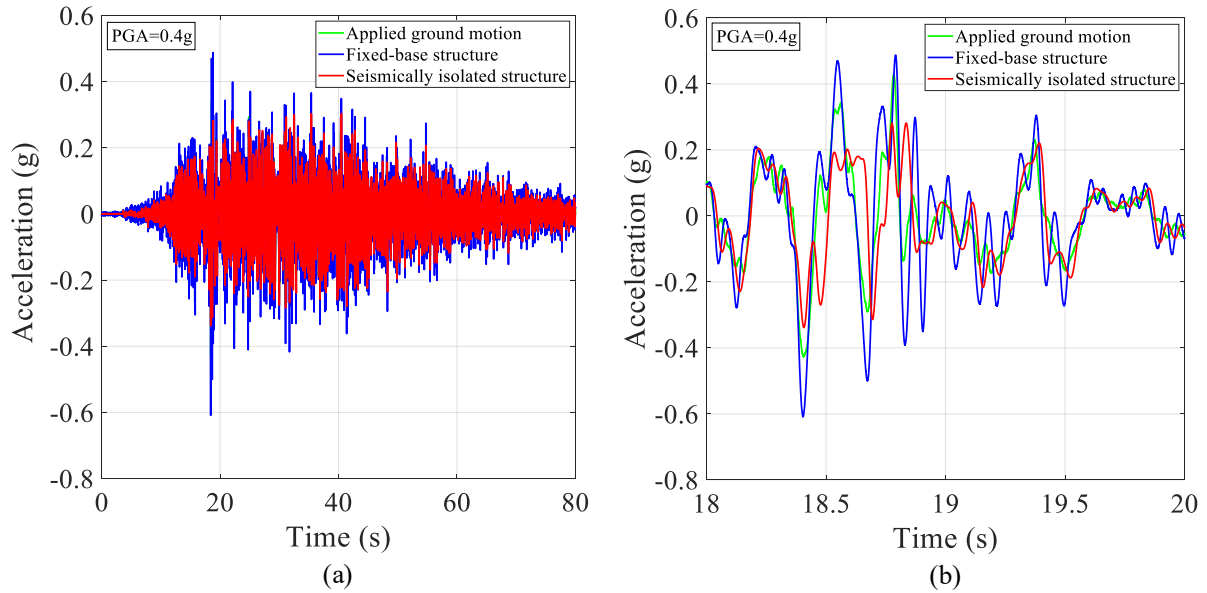


403  
 404 **Figure 13.** (a) Full and (b) magnified acceleration time history response of the steel structure (mean  
 405 value of M1X and M4X in Fig. 4) subjected to Chi-Chi 1999 ground motion record (No. 1 in Table 2)  
 406 for two configurations: 1) Fixed-base structure 2) Seismically isolated structure (PVC-s).

407 **INFLUENCE OF GROUND MOTION VARIATION**

408 The influence of the variation of the ground motion characteristics on the efficiency of the  
 409 proposed seismic isolation technique is quantified by subjecting the model structure to the

410 horizontal component of the Gorkha 2015 ground motion excitation (No. 3 in Table 2) and the  
 411 Kobe 1995 ground motion excitation (No 7 in Table 2), scaled to the same  $PGA=0.4g$ . As  
 412 shown Fig. 14 and 15, the maximum acceleration of the seismically isolated structure for the  
 413 two ground motions is  $0.6g/0.3g=2$  and  $0.44g/0.28=1.57$  times lower than the corresponding  
 414 fixed-base value.

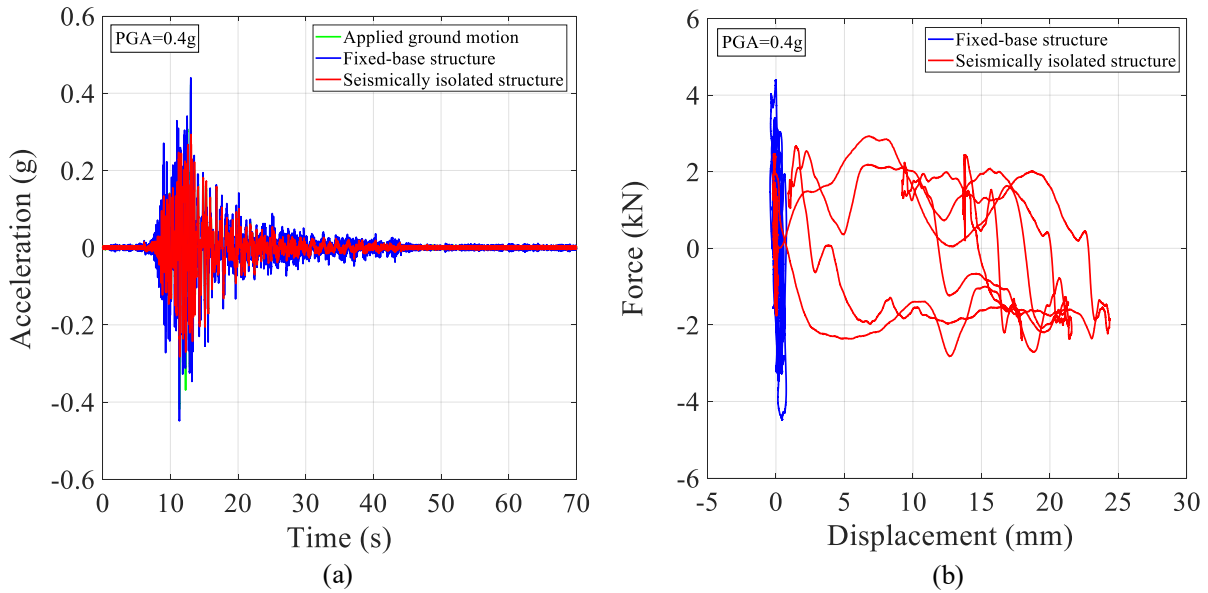


415  
 416 **Figure 14.** (a) Full and (b) magnified acceleration time history response of the steel structure (mean  
 417 value of M1X and M4X in Fig. 4) subjected to Gorkha 2015 ground motion record (No. 3 in Table 2)  
 418 for two configurations: 1) Fixed-base structure 2) Seismically isolated structure (PVC-s).

419 The hysteretic isolation force-sliding displacement response of the proposed PVC-s  
 420 isolation system to Kobe 1995 ground motion excitation is shown in Fig. 15b. The  
 421 corresponding force-displacement loop for the fixed-base superstructure subjected to the same  
 422 ground motion excitation is also shown in the figure. The manifestation of no inelastic behavior  
 423 and damage in the fixed-base superstructure subjected to a much higher force than the isolated  
 424 case is attributed to its high stiffness and strength. This high strength is chosen due to the need  
 425 for frequent crane transportation of the structure during the conduction of a large-scale  
 426 experimental parametric investigation.

427 The values of the mean friction coefficient  $\hat{\mu}$ , the maximum acceleration of the isolated  
 428 structure  $a_{isolated,max}$ , the maximum acceleration of the fixed-base structure  $a_{fixed-base,max}$ , the  
 429 maximum sliding displacement  $u_{sliding,max}$ , the residual displacement  $u_{residual}$  and the maximum  
 430 sliding velocity  $\dot{u}_{sliding,max}$  for all the ground motions considered in this study are summarized in  
 431 Table 4 and Fig. 16. As shown in Table 4, the mean value of the reduction ratio between the

432 acceleration of the isolated structure and the acceleration of the fixed-base structure is 2.24.  
 433 The high mean value of this ratio elucidates the efficiency of the proposed seismic isolation  
 434 system for the ground motion ensemble presented in this study.



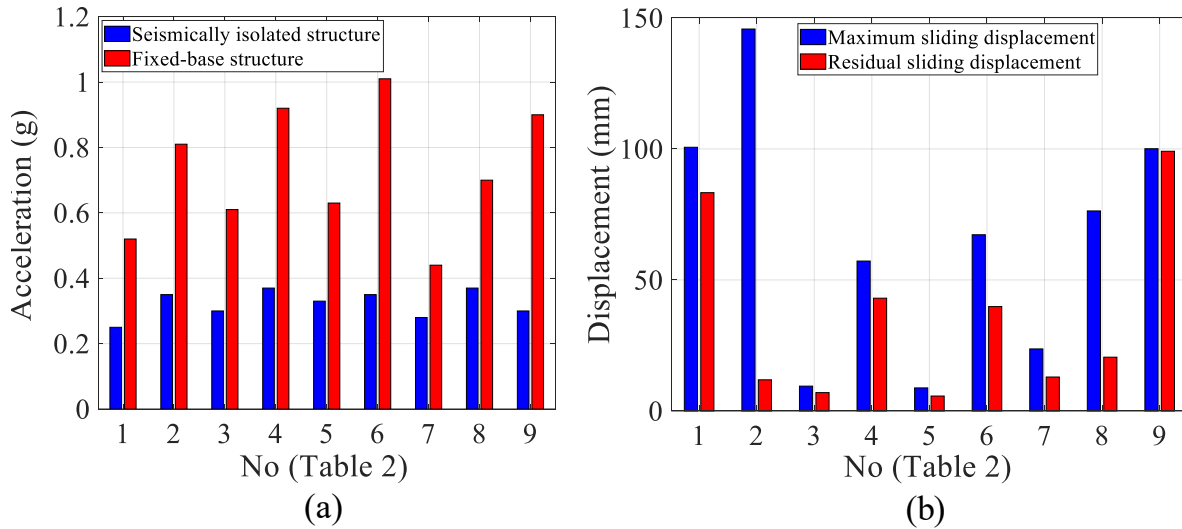
435  
 436 **Figure 15.** (a) Acceleration time history response and (b) hysteretic force-displacement loop of the  
 437 structure subjected to Kobe 1995 ground motion record (No. 7 in Table 2) for two configurations: 1)  
 438 Fixed-base structure 2) Seismically isolated structure (PVC-s).

439 **Table 4.** Mean and standard deviation values of the fundamental response parameters of the seismically  
 440 isolated and the corresponding fixed-base structure due to the ground motions presented in this study  
 441 (Ground motion ensemble shown in Table 2).

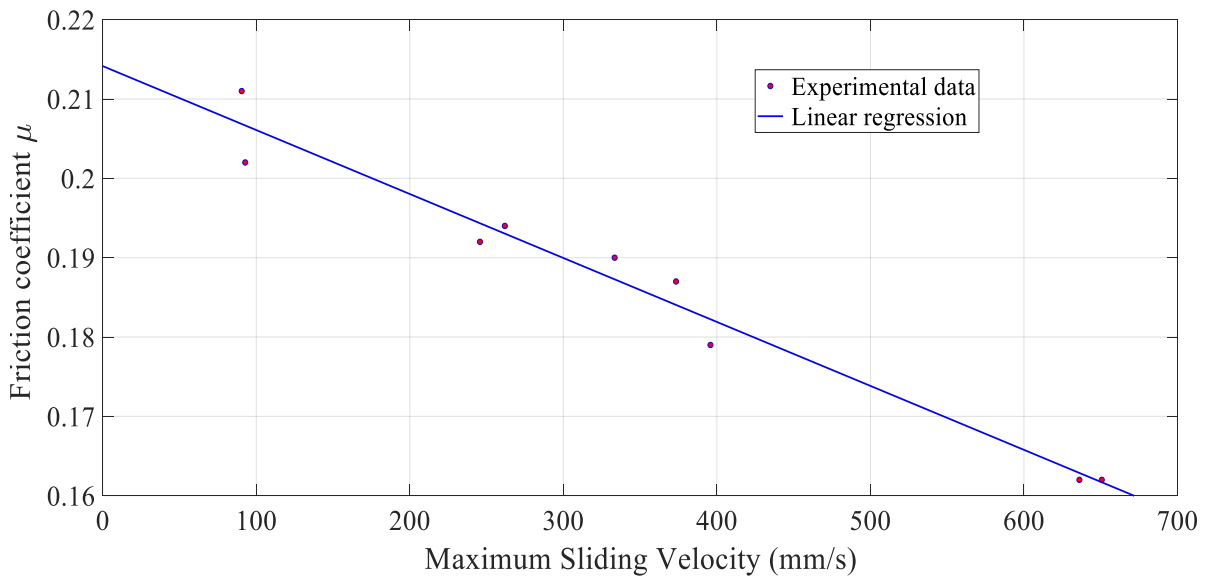
No.	PGA (g)	$\hat{\mu}$	$a_{isolated, max}$ (g)	$a_{fixed-base, max}$ (g)	$a_{isolated, max} / a_{fixed-base, max}$	$u_{sliding, max}$ (mm)	$u_{residual}$ (mm)	$\dot{u}_{sliding, max}$ (mm/s)
1	0.4	0.187	0.25	0.52	2.08	100.64	83.31	373.4
2	0.6	0.162	0.35	0.81	2.31	145.78	11.88	650.7
3	0.4	0.211	0.30	0.61	2.03	9.47	6.99	90.5
4	0.6	0.194	0.37	0.92	2.49	57.21	43.04	261.9
5	0.4	0.202	0.33	0.63	1.90	8.79	5.69	92.8
6	0.6	0.162	0.35	1.01	2.89	67.25	39.83	636.1
7	0.4	0.192	0.28	0.44	1.57	23.67	12.95	245.7
8	0.6	0.179	0.37	0.70	1.89	76.34	20.52	395.8
9	0.5	0.190	0.30	0.90	3.0	100.08	99.11	333.4
$\hat{\mu}$		0.187	0.33	0.73	2.24	65.47	35.92	340.12
$\sigma$		0.015	0.04	0.18	0.45	43.68	32.31	190.04

442  
 443 The corresponding mean values for the maximum sliding displacement during each ground  
 444 motion excitation  $u_{sliding, max}$  and residual displacement  $u_{residual}$  are 65.47mm and 35.92mm,

445 respectively. The mean value of the maximum sliding displacement indicates that the risk of  
 446 pounding of the structure against the surrounding gravel and soil is minimal for the ground  
 447 motion excitations investigated in this study. The low mean value of the residual displacement  
 448 shows that the recentering of the prototype structure using pulling forces from trucks might be  
 449 not required for many of the ground motions presented in this study (Fig. 16b) and the design  
 450 of flexible pipe connections could protect the services (water, sewage system) of the structure  
 451 from disruption.



452  
 453 **Figure 16.** (a) Maximum acceleration and (b) Sliding displacement response of the model structure  
 454 subjected to the earthquake records used in this study (Ground motion ensemble shown in Table 2).

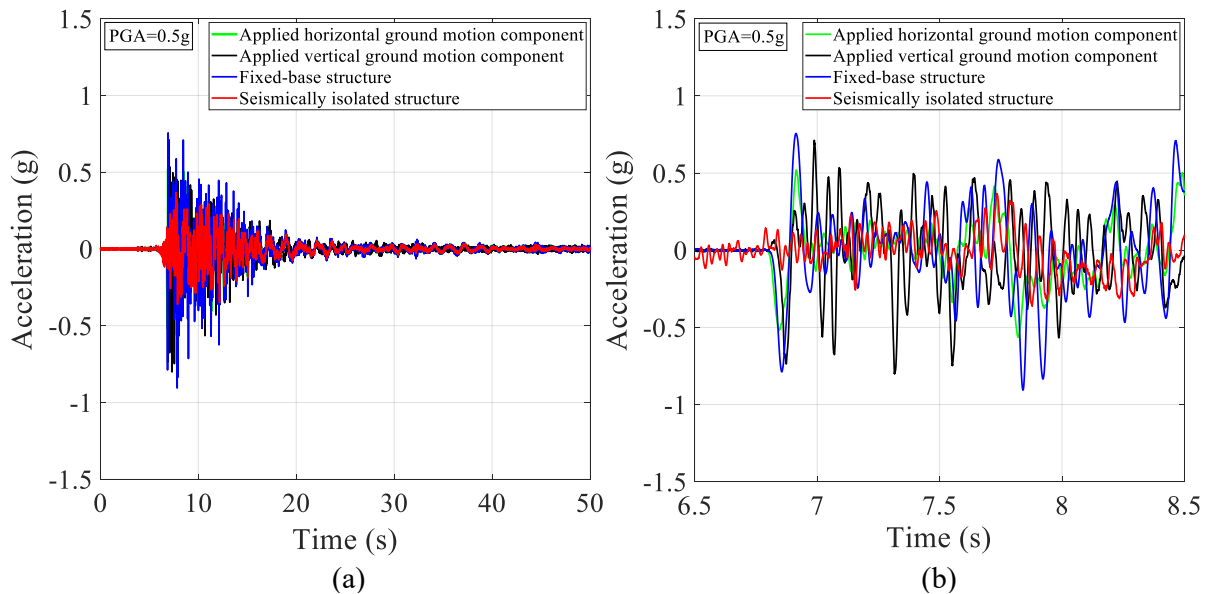


455  
 456 **Figure 17.** Friction coefficient  $\mu$  versus maximum sliding velocity of the model structure  
 457 subjected to the earthquake records used in this study (Ground motion ensemble shown in Table 2).

458 As shown in Fig. 17, the decrease of the mean value of the friction coefficient for increased  
 459 values of the maximum sliding velocity can be well approximated by a linear regression curve  
 460  $y=p_1x+p_2$  with  $p_1=-8.06 \cdot 10^{-5}$  and  $p_2=0.214$ . This linear decrease observed for maximum sliding  
 461 velocity values above 90mm/s is consistent with the corresponding decrease of the friction  
 462 coefficient value observed by Lomiento et al. (2013) for sliding bearings under seismic  
 463 excitation.

464 **INFLUENCE OF SIMULTANEITY OF GROUND MOTION COMPONENTS**

465 The effect of the simultaneity of the three components of a ground motion excitation on the  
 466 effectiveness of the presented PVC-s seismic isolation technique is determined through the  
 467 investigation of the response of the model structure to L’Aquila 2009 ground motion excitation,  
 468 scaled to a PGA=0.5g (No. 9 in Table 2). Furinghetti et al. (2017) highlighted the importance  
 469 of the consideration of the biaxial interaction of the horizontal components of a ground motion  
 470 excitation for the response of friction-based isolation systems due to the change in the  
 471 orientation of the frictional force during the motion compared to the case of a unidirectional  
 472 excitation. The destructive potential of vertical ground motion components during the L’Aquila  
 473 2009 earthquake has been presented by Verderame et al. (2011), Stewart et al. (2012) and  
 474 Zimmaro et al. (2018).



475  
 476 **Figure 18.** (a) Full and (b) magnified acceleration time history response of the steel structure (mean  
 477 value of M1X and M4X in Fig. 4) subjected to L’Aquila 2009 ground motion record (No. 9 in Table 2)  
 478 for two configurations: 1) Fixed-base structure 2) Seismically isolated structure (PVC-s).

479 The presence of a strong vertical ground motion component in the applied ground motion  
480 excitation did not inhibit the sliding behavior of the structure. As presented in Fig. 18, the  
481 acceleration reduction of the seismically isolated structure compared to the fixed-base case is  
482  $0.9g/0.3g=3$ . The response of the structure subjected to this ground motion excitation indicates  
483 a possible independence of the performance of the presented seismic isolation from the  
484 presence of vertical ground motion components. However, the independence observed for this  
485 ground motion should be verified by future shaking table tests exciting the structure with an  
486 ensemble of records with strong vertical ground motion components. The potential effect of  
487 the phasing of these components on the performance of the proposed system could be also  
488 quantified by future experimental or analytical studies.

## 489 **DESIGN REQUIREMENTS**

490 In light of the experimental results presented in this section, the proposed design  
491 requirements for the implementation of the proposed methodology in developing countries are:

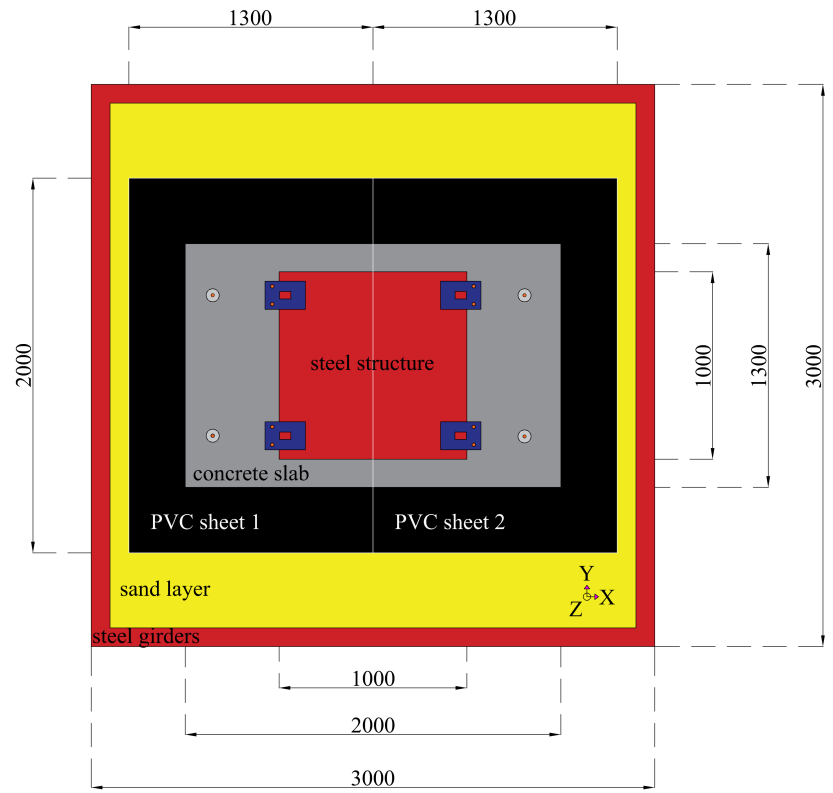
- 492 • The presence of a flat ground surface (inclination  $<5^\circ$ ) as a base for the construction  
493 of the PVC-s configuration.
- 494 • The use of 6mm thick, upper and bottom PVC sheets for the sandwich configuration  
495 which cover at the minimum the dimensions of the foundation slab of the structure.
- 496 • The deposition of 750g of sand per  $m^2$  of the ‘sandwiched’ PVC area with a 20cm-  
497 height drop from the bottom PVC surface using a 2.2mm sieve.

## 498 **EFFECT OF PVC SHEET DISCONTINUITIES**

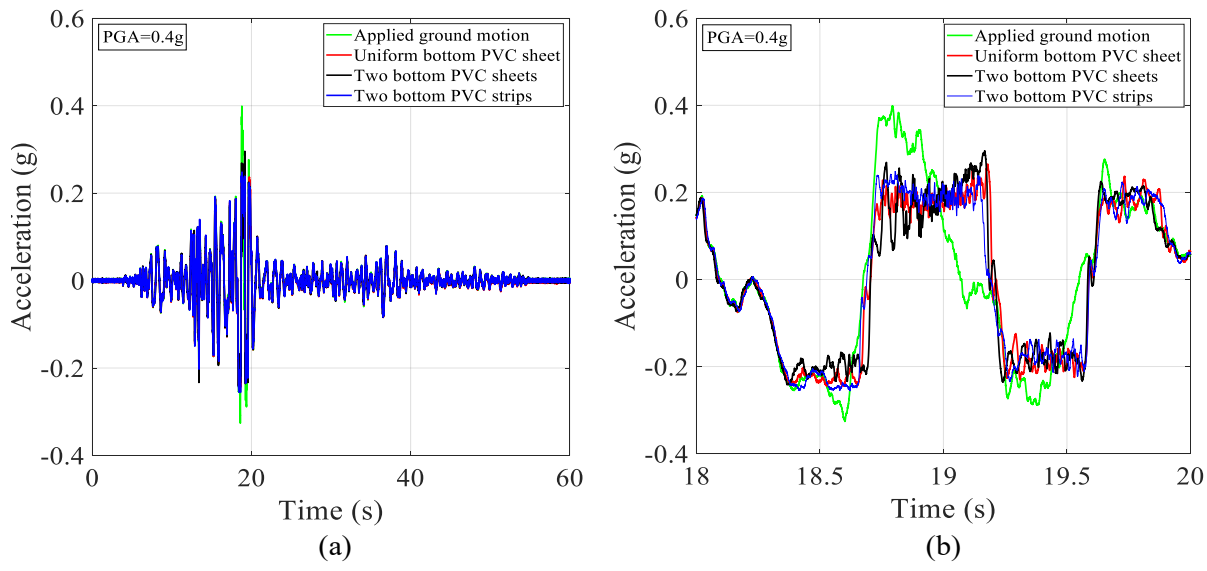
499 The constructability of the presented seismic isolation technique in developing countries is  
500 a prerequisite for its wide application as a novel and practical design solution in these countries.  
501 Within this context, the investigation of the influence of an alternative PVC sheet configuration  
502 that entails the placement of two bottom  $2m \times 1.3m$  PVC sheets next to each other and below  
503 the concrete slab (Fig. 19) is interesting for two main reasons: first, it simulates a practical  
504 case, where multiple PVC sheets of smaller size are placed as the bottom sandwich surface of  
505 the prototype structure, as these PVC sheets are more economical and can be easily transported.  
506 Second, this configuration allows for the quantification of the effect of the discontinuity formed  
507 in the contact zone between the two PVC sheets on the sliding performance of the PVC-s  
508 seismic isolation technique. As shown in Fig. 20, the maximum acceleration response on the



509 concrete slab and the associated frictional characteristics of the PVC-sand-PVC interface are  
 510 not significantly influenced by the presence of discontinuities in the bottom PVC sheet.



511  
 512 **Figure 19.** Overview of the configuration based on the use of two PVC sheets (Dimensions in mm).

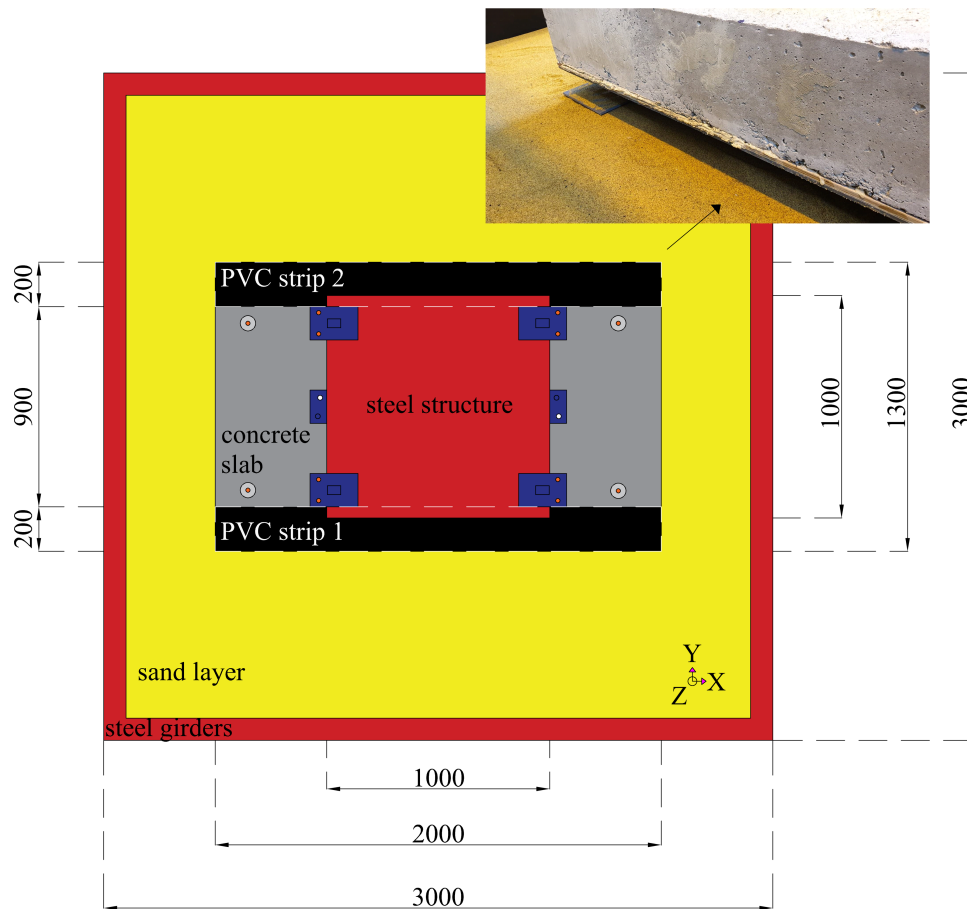


513  
 514 **Figure 20.** (a) Full and (b) magnified acceleration time history response of the concrete slab (mean  
 515 value of C1X and C4X in Fig. 4) subjected to Chi-Chi 1999 ground motion record (No. 1 in Table 2)  
 516 founded on two PVC sheets and comparison with the uniform PVC sheet configuration.

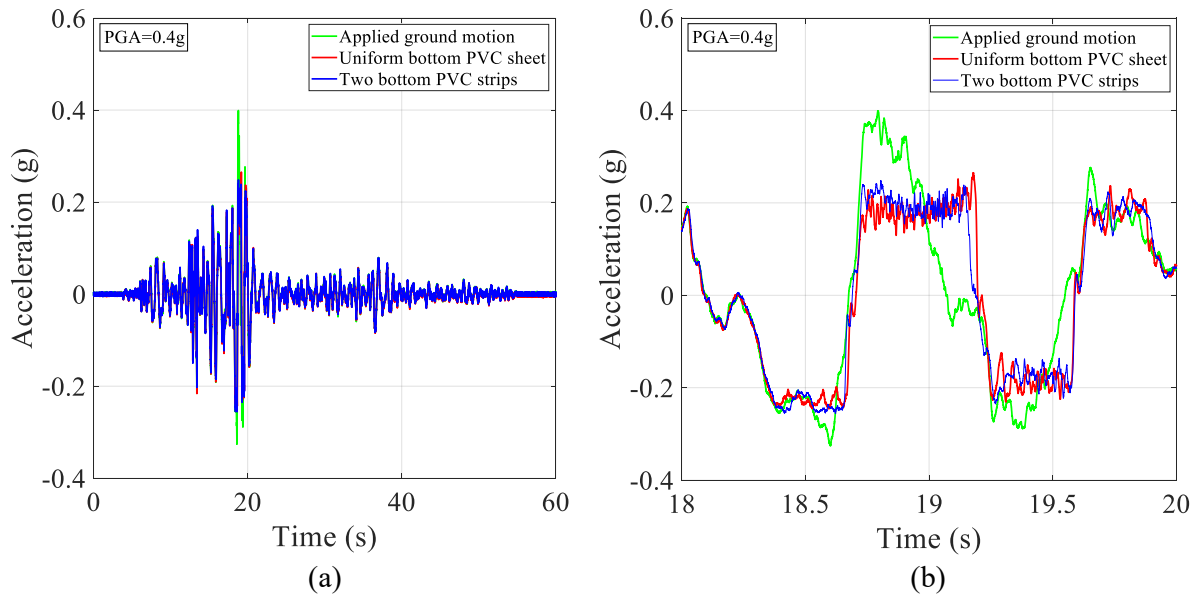
517 **IS THE WHOLE FOUNDATION SLAB NECESSARY?**

518 The potential decrease of the contact surface between the PVC sheets and the concrete slab  
519 is investigated using two bottom 2mx0.2m PVC strips below the concrete slab, as shown in  
520 Fig. 21. This PVC configuration represents a case where the foundation slab of the prototype  
521 structure is replaced by four concrete foundation beams (strip footings), thus further reducing  
522 the construction cost of the proposed seismic isolation strategy.

523 The experimentally derived frictional behavior of the structure founded on the presented  
524 PVC strip interface (Fig. 21) does not substantially change compared to the original uniform  
525 PVC sheet configuration, as illustrated in Fig. 22. This independence of the performance of the  
526 proposed system from the bottom PVC configuration guarantees the constructability of the  
527 presented seismic isolation technique for various foundation slab geometries and indicates a  
528 potential construction cost decrease through the replacement of the concrete foundation slab  
529 by four concrete foundation beams.



530  
531 **Figure 21.** Overview of the configuration based on the use of two bottom PVC strips (Dimensions in  
532 mm).



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**Figure 22.** (a) Full and (b) magnified acceleration time history response of the concrete slab (mean value of C1X and C4X in Fig. 4) subjected to Chi-Chi 1999 ground motion record (No. 1 in Table 2) founded on two PVC strips and comparison with the uniform PVC sheet configuration.

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## CONCLUSIONS

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The large-scale experimental investigation presented in this study focuses on the effect of an innovative, low-cost PVC ‘sand-wich’ (PVC-s) seismic isolation system towards seismic damage mitigation in developing countries. The robustness of this seismic isolation strategy is attributed to the presence of two sliding mechanisms: The fundamental seismic protection mechanism, characterized by the relative sliding between two PVC sheets with encapsulated sand grains, is activated for ground motions of moderate earthquake intensity, corresponding roughly to a PGA of 0.25g. The optimal sand surface density range between 40 and 750g/m<sup>2</sup> that leads to the presented sliding behavior is experimentally determined. Within this density range, the sand surface density value of 750g/m<sup>2</sup> is recommended in this study as it renders the lowest dispersion of the friction coefficient. A secondary mechanism, triggered by the sliding behavior of a PVC surface against a sand foundation layer below the bottom PVC can be activated for ground motions of higher earthquake intensity, exceeding a PGA=0.35g.

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The repeatability and the efficiency of the presented seismic isolation technique for subsequent ground motions of different characteristics and alternative PVC sheet configurations is experimentally validated. The beneficial effect of the PVC-s seismic isolation strategy on the reduction of seismic accelerations and forces is shown through a comparison with the experimentally derived response of the corresponding fixed-base structure.

555 The activation of the sliding behavior of the structure using the proposed seismic isolation  
556 for a ground motion containing a strong vertical component is presented. Nevertheless, the  
557 conduction of future large-scale tests exploring the performance of the proposed system for a  
558 wide range of ground motion records with vertical components is required for the  
559 generalization of the trends observed for this ground motion. The frictional characteristics of  
560 the proposed interface are also found to be preserved even for the case of a placement of the  
561 bottom PVC layer in two separate pieces, which shows the constructability of the proposed  
562 seismic isolation technique in countries with transportation of PVC availability issues. A  
563 reduction of the bottom PVC surface using PVC strips yielded equivalent results with the case  
564 of a uniform bottom PVC sheet, thus indicating a potential, more economical replacement of  
565 the bottom concrete slab by four concrete beams (strip footings) based on PVC strips.

566 The presence of residual horizontal displacements of a maximum amplitude of 10cm and a  
567 mean amplitude of 3.6cm is expected after the occurrence of strong ground motion excitations.  
568 However, the low amplitude of these expected displacement estimates allows for a relatively  
569 simple recentering of the displaced low-rise structure using pulling forces from trucks or other  
570 types of vehicles.

571 Inevitably, there are some inherent limitations in the findings derived from the large-scale  
572 investigation presented in this study. First, the sliding response observed for the geometry of  
573 the selected test structure simulating a typical masonry school located in Nepal should be  
574 generalized to other geometries by future experimental studies. Second, the experimentally  
575 convenient choice of a steel test structure of higher strength compared to an alternative masonry  
576 test structure cannot shed light on the seismic damage of a fixed-base masonry structure.  
577 Therefore, the efficiency of the holistic hybrid design approach presented in this study will be  
578 further verified by a large-scale shaking table investigation of a scaled-down masonry model  
579 of the prototype structure at the shaking table of University of Bristol.

580

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