Isolated soil mass at foundation for mitigating seismic risk

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The installation of passive structural systems to protect existing buildings from seismic risk is very common, yet it can be expensive and is not always feasible in the case of valuable buildings. It may affect the integrity of structures of artistic or historical relevance in a way that cannot be accepted in the light of the most recent rules of conservation and retrofitting. As an alternative, ground improvement techniques can be used to modify ground properties, in order to mitigate the intensity of shaking at ground level. This paper focuses on the case of a V-shaped barrier that may be formed by assembling inclined and partially overlapped columns of artificially softened soil to isolate a mass in the ground. The response of a single degree of freedom founded within the isolated ground mass is then studied by way of finite-element analyses and the results are commented on.

Notation

$a_{\max, \text{IM}}$	peak acceleration of an isolated soil mass
$a_{\max, \text{SDOF_IM}}$	peak acceleration of a single-degree-of-freedom system founded within the isolated mass
D_0	small-strain damping of sand
E	Young's modulus
f_{IM}	natural frequency of a single-degree-of-freedom system founded on untreated ground layer
f_s	first natural frequency of the ground layer
f_{SDOF}	natural frequency of an isolated soil mass
G	shear modulus
H_{IM}	depth of an isolated soil mass
I_{ASDOF}	Arias intensity of the time histories of acceleration of a single-degree-of-freedom system founded on untreated ground layer
$I_{\text{ASDOF_IM}}$	Arias intensity of the time histories of acceleration of a single-degree-of-freedom system founded on an isolated soil mass
I_s	spectral intensity of untreated soil layer
I_{SIM}	spectral intensity of an isolated soil mass
k	stiffness
L_{IM}	base of an isolated soil mass
m	mass
n	shape coefficient to calculate the volume of the isolated mass

S	thickness of a soft barrier
$S_v(\xi, T)$	spectral velocity for a given structural damping ratio
T_1, T_2	two structural periods defining an integration range
V_s	velocity of shear waves
x	horizontal distance from control point
z	depth from ground surface
α_R, β_R	Rayleigh damping coefficients
β	inclination of the V-shaped barrier sides about the horizontal axis
η	dynamic impedance
ξ	structural damping ratio
ρ	density

Introduction

As far as the seismic risk of new buildings is concerned, refined technical elements able to dissipate most of the seismic energy (such as seismic isolators and dampers) have been developed in structural engineering. In contrast, the seismic protection of existing buildings is still a matter of great concern. In most countries of southern Europe and the Mediterranean area, where the seismic hazard is high, the buildings in the historic centre of most towns are hundreds of years old. Hence, the protection of both population and cultural heritage becomes a crucial issue (e.g. D'Ayala and Ansal, 2013).

The foremost solution to tackle seismic risk for existing structures is structural retrofitting – that is, the use of structural or ground modifications aimed at reducing the vulnerability. Typical retrofitting procedures, such as passive structural systems installed at the foundation level of existing buildings, tend to modify the capacity curve to meet the seismic demand with the needed safety factor. However, installation procedures are expensive and not always feasible, as for instance in the case of valuable buildings. Here they may affect the integrity of structures of artistic or historical relevance in a way that cannot be accepted in the light of the most recent rules of conservation and retrofitting, as stated in the reference documents of Unesco (the United Nations Educational, Scientific and Cultural Organization) (e.g. ICOMOS, 1964). As an alternative, the seismic risk can be reduced by way of a reduction of the hazard, leading to a decrease of the seismic demand. Ground improvement techniques can be used to modify ground properties, in order to mitigate the intensity of shaking at ground level.

The propagation of shear waves in the ground can be modified by introducing a layer of artificially modified material (for instance a grouted layer), having a lower dynamic impedance (defined as $\eta = \rho V_s$, where ρ is the density of the material and V_s is the velocity of shear waves) than the natural soil.

Some authors have recently explored the possibility of implementing a ‘soft barrier’ – more feasible from a technical point of view – to protect existing buildings from earthquakes (e.g. Dietz and Muir Wood, 2006; Muir Wood, 2006; Tsang *et al.*, 2012). A very interesting contribution comes from Kirtas and Pitilakis (2009) and Kirtas *et al.* (2009): in their papers they discuss the results of a series of numerical analyses of different geometrical schemes of the isolating barrier. The building is assimilated to a single-degree-of-freedom (SDOF) system. Following the approach of Kirtas and Pitilakis (2009), Lombardi (2014) and Flora *et al.* (in the unpublished (at time of writing) paper, Numerical analyses of the effectiveness of soft barriers into the soil for the mitigation of seismic risk, by Flora A, Lombardi D, Nappa V and Bilotta E) analysed the effectiveness of similar barriers also on the basis of the results of laboratory tests on compounds that can be grouted in the soil to form the barriers. The numerical schemes analysed in these works were in fact supported by a study on the feasibility of the barrier using peculiar grouts, having very low density, stiffness and shear strength. Possible grouts showing such features were tested in the laboratory, and their mechanical properties were published by Flora *et al.* (2015). According to that work, the barrier can be effective without causing excessive settlements, provided that the soft layer is deep enough and is created at a distance at least equal to the foundation width below the foundation level.

Centrifuge tests on reduced-scale models of similar barriers were recently published by Nappa *et al.* (2016), showing their effectiveness in reducing the ground shaking at the surface within an isolated mass of soil.

This paper focuses on the case of a V-shaped barrier that may be easier to form by assembling inclined and partially overlapped columns, for instance through a deep mechanical mixing procedure. Starting from the back-analysis of the results of free-field centrifuge tests, a series of numerical analyses have been carried out. A series of different input signals were applied to the base of the numerical model. Each signal was extracted from databases of recorded natural events, and a wide range of frequency content, duration and shaking amplitude was covered. The response of an SDOF founded within the isolated ground mass was then observed and commented on.

Isolated soil mass

The isolated soil mass can be created in different shapes. Lombardi *et al.* (2013, 2014) and Nappa *et al.* (2016) showed in both numerical and experimental analyses the effectiveness of a deep horizontal layer of soft soil to reduce the ground shaking at the surface. In their unpublished paper, Flora *et al.* studied a similar treatment in the form of a rectangular ‘caisson’ with soft walls and base, also suggesting as an alternative a V-shaped treatment (Figure 1(a)). The latter is easier to form on site with inclined and partially overlapped drillings (Figure 1(b)).

As long as the barrier is soft enough to prevent significant deformations within the isolated soil mass, it is reasonable to assume that such a mass (m) will undergo an almost rigid-body motion, as shown by Flora *et al.* in their unpublished paper using numerical analyses. Then, the isolated mass itself can be considered as an SDOF system, whose stiffness k is mostly ruled by the soft barrier’s geometrical and mechanical properties. Based on these considerations, the unpublished paper by Flora *et al.* proposes a simplified equation to calculate the natural frequency f_{IM} of the isolated mass, assumed as an SDOF system. For the cases of a rectangular (of depth H_{IM} , base L_{IM} and thickness of the soft barrier S) and a V-shaped caisson (of maximum depth H_{IM} , base L_{IM} and thickness of the soft barrier S), the expressions of the natural frequency are respectively

$$1a. \quad f_{IM} = \frac{1}{2\pi} \left[\frac{1}{\rho} \left(\frac{2E}{SL_{IM}} + \frac{G}{SH_{IM}} \right) \right]^{0.5} \text{ rectangular caisson}$$

Figure 1. (a) Sketch of possible layout and (b) main geometrical features of a plane strain model

$$f_{IM} = \frac{1}{2\pi} \left\{ \frac{4}{\rho^2} \left[\left(\frac{2E}{SL_{IM}} \right)^2 + \left(\frac{G}{SH_{IM}} \right)^2 \right] \right\}^{0.25}$$

1b. V-shaped caisson

where E and G are the Young's and the shear modulus of the soft soil isolating the mass and ρ is the isolated soil density.

It is worth noting that for low values of E and G , as well as for a large isolated soil mass ($m = n\rho H_{IM}L_{IM}$, with $n = 1$ for the

rectangle or $n = 0.5$ for the triangle), f_{IM} is generally much lower than the first natural frequency of the ground layer, f_s . Because of such a 'filtering' action at the foundation level, the soft barrier seems suited for protecting structures with high natural frequencies, such as squat buildings.

This is consistent with the results presented in their unpublished paper by Flora *et al.*, which showed that a very shallow V-shaped barrier in an elastic-perfectly plastic ground layer would be ineffective, or even detrimental, depending on the opening angle β . In contrast, a deeper V-shaped caisson can be effective for the attenuation of high-frequency signals. These authors also showed

(a)

(b)

Figure 2. (a) Benchmark centrifuge model (adapted after Nappa *et al.* 2016) and (b) numerical mesh

E_{50}^{ref} : kN/m ²	$E_{\text{oed}}^{\text{ref}}$: kN/m ²	$E_{\text{ur}}^{\text{ref}}$: kN/m ²	m	c : kN/m ²	ϕ : °	ψ : °	ν	ρ^{ref} : kN/m ²	K_0^{nc}	E_0^{ref} : kN/m ²	$\gamma_{0.7}$
30 000	30 000	90 000	0.55	0	42	16	0.25	100	0.4	270 000	0.0002

Reference pressure p_{ref} is 100 kPa

Table 1. Hardening soil small-strain parameters adopted for the sand

that lower inclinations of the sides on the horizontal provide better efficiencies in the control point CP. Larger and deeper soft V-shaped isolate masses seem therefore the most efficient ones.

This work then focuses on a geometrical layout ($H_{\text{IM}} = 12$ m, $\beta = 45^\circ$) that can be considered a good compromise in terms of effectiveness and on-site feasibility.

Numerical analyses

Numerical simulations were performed using the finite-element (FE) code Plaxis 2D (Brinkgreve *et al.*, 2011). The numerical model was calibrated by using the results of centrifuge testing on a reduced-scale model by Nappa *et al.* (2016). A sketch of the centrifuge model is shown in Figure 2(a). The ground layer was made of dry Hostun sand, and the V-shaped barrier was created therein through latex balloons filled with a hydrogel with low shear stiffness and strength (Flora *et al.*, 2015).

The numerical mesh is represented in Figure 2(b). Boundary conditions reproduced the laminar box used in the centrifuge tests: periodic boundary at the lateral sides, where displacements between corresponding nodes were restrained, and simple supports at the base, simulating a reflective boundary. The numerical analyses were carried out at prototype scale, corresponding to a box of 40×22.8 m as shown in Figure 2.

Since both the stiffness decay and the hysteretic damping of the soil are crucial in the prediction of the stress–strain behaviour under cycling, as it is in the case of shaking due to earthquakes, a suitable constitutive model needed to be adopted in the numerical analyses. Hence, to model a realistic behaviour of the ground, allowing for stress and strain dependence of stiffness and capturing the hysteretic behaviour of sand and the associated hysteretic damping in unloading–reloading cycles, Hardening Soil with small-strain overlay was used (Benz *et al.*, 2009; Schanz *et al.*, 1999). This is a constitutive model implemented in the Plaxis

ρ : kN/m ³	G : kN/m ²	ϕ : °	ν	V_S : m/s
1020	109	10	0.4	12

Table 2. Mohr–Coulomb parameters adopted for the soft barrier

code that takes into proper account strain hardening plasticity and small-strain behaviour of sand.

The model parameters for HN31 Hostun sand are reported in Table 1. They were partly extracted from literature (Benz, 2007) and partly back-figured from the results of centrifuge tests as shown by Nappa *et al.* (2016). The small-strain damping of the sand ($D_0 = 0.5\%$) was modelled through the Rayleigh formulation, by estimating the coefficients α_R and β_R by using the ‘double-frequency approach’ suggested by Park and Hashash (2004). To achieve this aim, the predominant frequency of the input signals and the first natural frequency of the soil layer were used.

The soft barrier was modelled as an elastic-perfectly plastic Mohr–Coulomb material. Its shear strength was determined through shear tests (Flora *et al.*, 2015), while the elastic shear wave velocity was obtained by means of air-hammer pulse tests carried out during the centrifuge flight (Nappa *et al.*, 2016). The adopted values of the parameters are summarised in Table 2. The

Figure 3. Recorded and calculated time histories of acceleration and Fourier spectra: (a) model scale; (b) prototype scale

small-strain damping of the barrier (D_0) was derived from back-analysis and was assumed equal to 2.5%.

To check the reliability of the numerical model, the centrifuge tests with V-shaped barrier were first simulated at model scale. Figure 3(a) shows, as an example, the results of such calculations for one signal, compared with the experimental result obtained from centrifuge test in the control point. It can be observed that the calculated and recorded acceleration time histories and the corresponding Fourier spectra are in good agreement. The same calculation was then carried out at prototype scale, thus validating the numerical model (see Figure 3(b)).

Hence, the numerical calculations were performed by using the mesh shown in Figure 2(a) with natural signals.

The time histories of acceleration used as base input motions at the bottom boundary of the FE mesh are shown in Figure 4 (all scaled to the same value of amplitude, that is 0.3g). The relevant Fourier spectra are also shown in the figure. Table 3 reports the main characteristics of each record. It is worth noting that the input signals were chosen to cover the broadest possible range of frequency contents (approximately 0.5–10 Hz). In such a way, the potential beneficial or detrimental effects of the isolating barrier can be better investigated, and its limitations considered.

Figure 4. Time history of acceleration (left panels) and Fourier spectra (right panels) of the adopted input signals (cf. Table 3)

Database	ID	Name	Date	PGA: m/s ²	Arias intensity: m/s	Predominant period: s	Predominant frequency: Hz	Significant duration: s
ITACA	146	Campano Lucano	23/11/1980	3.17	1.35	0.2	5.0	38.5
ESD	175	Lazio Abruzzo	07/05/1984	1.23	1.30	0.4	2.5	12
	1635x	South Iceland	17/06/2000	1.53	0.87	0.24	4.2	4.5
	1635y	South Iceland	17/06/2000	1.29	0.73	0.16	6.3	5.8
	1885	Kalamata	13/10/1997	1.15	1.93	0.3	3.3	17.7
	2142x	South Iceland (aftershock)	21/06/2000	1.13	1.23	0.3	3.3	5.3
Japanese Kyoshin	SZO002	Japan	1997	1.67	0.89	0.3	3.3	4.2
Network database	EW 83	Kobe	17/01/1995	3.02	1.35	0.08	12.5	15

PGA, peak ground acceleration; ITACA, Italian Accelerometric Archive; ESD, European Strong-Motion Database

Table 3. Time histories of acceleration used as input motions in FE analyses

Numerical results

The numerical analyses confirmed that the insertion of a soft barrier completely modifies the motion of the soil within the protected volume.

For all the input signals, the peak acceleration $a_{\max,IM}$ at CP is shown in Figure 5. Figure 5 also reports the distribution of $a_{\max,IM}$ with depth as it results from a parametric study by Lombardi *et al.*

Figure 5. Normalised maximum acceleration along central vertical for all input signals: comparison with Lombardi *et al.* (2015)

(2015) on a similar scheme in an elastic-perfectly plastic ground layer. In that study, the input signals were the so-called Ricker wavelets (Ryan, 1994) of nominal frequency ranging between 0.5 and 5 Hz. This range covers the same range of significant frequency content of the natural signals used in this work.

In the figure, both z and $a_{\max}$ are normalised, dividing them respectively by H_{IM} (the maximum depth of the V barrier,

Figure 6. Normalised maximum acceleration at ground surface (all input signals): comparison with Lombardi *et al.* (2015)

Figure 1) and by the acceleration $a_{\max}$ at the considered depth without any barrier. The distribution of the same dimensionless ratio at the ground surface is shown in Figure 6, plotted along the normalised distance from the control point CP ($x \tan \beta / H_{\text{IM}}$).

The isolating effect of the soft barrier on the bounded mass of soil is evident in Figure 5, where the peak accelerations calculated above the barrier for the input signal with a predominant frequency higher than 2 Hz (see Table 3) are lower than the corresponding ones calculated without barriers. Outside the isolated mass (i.e. $z/H_{\text{IM}} > 1$ in Figure 5 and $x \tan \beta / H_{\text{IM}} > 1$ in Figure 6), the peak acceleration is almost unchanged, with the exception of a small portion of soil immediately beneath the barrier where the values are slightly higher because of wave reflection.

Figure 7 shows the results of the analyses in terms of the ratio between the maximum acceleration of an SDOF system founded within the isolated mass, $a_{\max_SDOF_IM}$, and that of the same SDOF founded on untreated ground, $a_{\max_SDOF}$, plotted against the SDOF characteristic frequency f_{SDOF} .

The natural frequency of the isolated mass, f_{IM} , is also shown in Figure 7 as a dashed vertical line. It was calculated according to Equation 1b, where the mechanical parameters of the soft barrier (E , G) and its density ρ can be obtained from Table 2, $H_{\text{IM}} = 12$ m, $L_{\text{IM}} = 24$ and $S = 0.8$ m.

It can be noticed that the average value of the ratio $a_{\max_SDOF_IM} / a_{\max_SDOF}$ for the eight input signals is below unity for SDOF systems with a natural frequency higher than about 2.4 Hz. This value is larger than that of the isolated soil mass (from Equation 1b, $f_{\text{IM}} = 1.4$ Hz).

In Figure 8 the Arias intensity of the time histories of acceleration of the SDOF system founded on the untreated ground layer, I_{ASDOF} , is plotted against the SDOF system frequency f_{SDOF} .

Figure 7. Ratio between maximum accelerations of an SDOF system founded within the isolated mass and the same SDOF system founded on the untreated ground

Figure 8. Arias intensity of the time history of the acceleration of an SDOF system founded on the untreated ground layer

As expected, SDOF systems with a natural frequency of at least 10 Hz would experience a high-intensity level of shaking if founded in the untreated ground layer undergoing the seismic excitations shown in Figure 4, having a wide frequency content in the same range.

In Figure 9 a similar plot shows the Arias intensity of the time histories of acceleration of the SDOF system founded within the isolated mass, $I_{\text{ASDOF_IM}}$. When comparing Figure 9 with Figure 8, the filtering effect of the isolated mass of soil at the SDOF foundation can be clearly observed, since the peaks of the Arias intensity are attracted around the natural frequency of the isolated mass (calculated by using Equation 10). Hence, in the same conditions of input motion at the base of the ground layer, only SDOF systems with a natural frequency in the narrow range 1–2.5 Hz would suffer high-intensity shaking.

In Figure 10 the ratio $I_{\text{ASDOF_IM}} / I_{\text{ASDOF}}$ is plotted along f_{SDOF} . The average function (shown as a bold dashed line) clearly indicates that the energy content of the SDOF system motion can

Figure 9. Arias intensity of the time history of the acceleration of an SDOF system founded within the isolated mass

Figure 10. Ratio between the Arias intensity of the time history of the acceleration of an SDOF system founded within the isolated mass and the same SDOF system founded on the untreated ground

be largely attenuated by the isolated foundation, again with the exception of SDOF systems with a natural frequency around that of the isolated soil mass, f_{IM} , also shown in the figure.

A synthetic and meaningful parameter for controlling the overall performance of an SDOF system is the well-known spectral intensity I_S (Housner, 1952), which allows the quantification of the free-field seismic response. I_S is defined as

$$2. \quad I_S(\xi) = \int_{T_1}^{T_2} S_V(\xi, T) dT$$

where $S_V(\xi, T)$ is the spectral velocity for a given structural damping ratio ξ (5% in this case) and (T_1, T_2) are two structural periods defining the integration range. Since this study is oriented toward the protection of squat masonry buildings, the two periods have been taken as equal to 0.1 and 0.5 s respectively. This choice implies that the height of building would range between 3 and 20 m (CEN, 2004).

Figure 11. Maximum spectral intensity ratio I_{SIM}/I_S : comparison with Lombardi *et al.* (2015)

The spectral intensity calculated for each input signal at the control point CP in free-field conditions within the isolated mass, I_{SIM} , has been normalised by the corresponding value I_S of the untreated soil layer. In Figure 11 the ratio I_{SIM}/I_S is plotted against the predominant frequency of the input signal (cf. Table 3). Figure 11 also displays the results of a parametric study by Lombardi *et al.* (2015) on a similar scheme under different assumptions for the behaviour of the ground layer.

Despite different kinds of input signals being used in that work (i.e. artificial Ricker wavelets), the dependency of the efficiency of the barrier on the frequency content of the input signal is very similar. Furthermore, the comparison confirms that the attainment of yielding conditions within the soft layers largely helps in increasing the effectiveness of the isolating barrier.

The filtering effect of the isolated mass around the foundation of an SDOF system, already observed in Figure 10, is implicit in Figure 11, since the Housner spectral intensity I_S has been calculated over the range of periods 0.1–0.5 s, corresponding to 2–10 Hz, which is far from the frequency of the isolating mass, f_{IM} . Therefore, the ratio I_{SIM}/I_S is always lower than unity.

Conclusions

Reducing seismic demand can conveniently protect structures with high natural frequency, namely squat buildings.

Soft barriers that isolate a mass of ground well beneath the foundation level can be created to achieve this aim by modifying the soil properties by using the same techniques usually adopted for ground improvement. Consequently, seismic site amplification can be locally modified.

The isolated mass of ground tends to shift the amplification of the input motion at the base of the structure towards its own frequency, thus reducing the seismic demand on those structures with larger natural frequencies.

Hence, whatever the frequency content of the seismic event is, a structure founded within the isolated mass will be efficiently protected if its fundamental frequency (or frequencies, in the case of a multiple-degree-of-freedom system) is (are) far enough from the fundamental frequency of the isolated mass of ground. From a design point of view, this means that the geometry of the isolated soil mass, as well as the properties of the soft grout to be adopted, must be carefully chosen to maximise this difference. To achieve this aim, a preliminary design may be based on Equations 1a and 1b. The effect of site amplification of the ground layering outside the barrier is somehow overridden.

Acknowledgements

The research activity was carried out at the University of Naples Federico II as part of the national research project PON_03 METRICS and the regional research and innovation project

TICISI. The financial support of the consortium Stress s.c.a r.l. is kindly acknowledged.

REFERENCES

- Benz T (2007) *Small Strain Stiffness of Soil and its Numerical Consequences*. PhD thesis, Universitat Stuttgart, Stuttgart, Germany.
- Benz T, Vermeer PA and Schwab R (2009) A small-strain overlay model. *International Journal for Numerical and Analytical Methods in Geomechanics* **33(1)**: 25–44.
- Brinkgreve RBJ, Swolfs WM and Engine E (2011) *PLAXIS User's Manual*. PLAXIS bv, Delft, the Netherlands.
- CEN (Comité Européen de Normalisation) (2004) EN 1998-1:2004: Eurocode 8: Design of structures for earthquake resistance – part 1: general rules, seismic actions and rules for buildings. CEN, Brussels, Belgium.
- D'Ayala D and Ansal A (2013) Local site conditions and seismic risk assessment of historic buildings. In *Geotechnical Engineering for the Preservation of Monuments and Historic Sites* (Bilotta E, Flora A, Lirer S and Viggiani C (eds)). CRC, London, UK, pp. 45–56.
- Dietz MS and Muir Wood D (2006) Shake table testing of a soft caisson for geotechnical seismic retrofit. In *Physical Modelling in Geotechnics: 6th ICPMG, Hong Kong* (Ng CWW, Zhang LM and Wang YH (eds)). Taylor & Francis, London, UK, vol. 1, pp. 501–506.
- Flora A, Bilotta E, Lirer S, Lombardi D and Nappa V (2015) Behavior of a sand-polyacrilate mixture. *Proceedings of the Sixth International Symposium on Deformation Characteristics of Geomaterials*, 15–18 Nov., Buenos Aires, Argentina.
- Housner GW (1952) Spectrum intensity of strong-motion earthquakes. In *Proceedings of the Symposium on Earthquakes and Blast Effects on Structures*. Earthquake Engineering Research Institute, Oakland, CA, USA, pp. 21–36.
- ICOMOS (International Council on Monuments and Sites) (1964) The Venice Charter: international charter for the conservation and restoration of monuments and sites. In *Proceedings of the 2nd International Congress of Architects and Technicians of Historic Monuments*. ICOMOS, Paris, France, pp. 25–31. See http://www.icomos.org/charters/venice_e.pdf (accessed 09/04/2016).
- Kirtas E and Pitilakis K (2009) Subsoil interventions effect on structural seismic response – part II: parametric investigation. *Journal of Earthquake Engineering* **13(3)**: 328–344.
- Kirtas E, Rovithis E and Pitilakis K (2009) Subsoil interventions effect on structural seismic response – part I: validation of numerical simulations. *Journal of Earthquake Engineering* **13(2)**: 155–169.
- Lombardi D (2014) *An Innovative Ground Treatment Approach for Seismic Risk Mitigation of Existing Structures*. PhD thesis, University of Naples Federico II, Naples, Italy.
- Lombardi D, Flora A, Lirer S and Silvestri F (2013) An innovative approach for reducing the seismic risk of existing buildings and historic sites. In *Geotechnical Engineering for the Preservation of Monuments and Historic Sites* (Bilotta E, Flora A, Lirer S and Viggiani C (eds)). CRC, London, UK, pp. 507–514.
- Lombardi D, Flora A, Lirer S and Silvestri F (2014) Soil grouting for the seismic protection of existing buildings. *Ground Improvement* **167(GI3)**: 206–218.
- Lombardi D, Bilotta E and Flora A (2015) Soft grouting for the mitigation of seismic risk. *Proceedings of XVI ECSMFE on Geotechnical Engineering for Infrastructure and Development*, Edinburgh, Scotland.
- Muir Wood D (2006) Modelling of dynamic soil problems. In *Proceedings of the 4th International Conference on Earthquake Geotechnical Engineering – Invited Lectures* (Pitilakis KD (ed.)). Springer, New York, NY, USA, pp. 131–149.
- Nappa V, Bilotta E, Flora A and Madabhushi G (2016) Centrifuge modelling of the seismic performance of soft buried barriers. *Bulletin of Earthquake Engineering*, <http://dx.doi.org/10.1007/s10518-016-9912-9>.
- Park D and Hashash YMA (2004) Soil damping formulation in nonlinear time domain site response analysis. *Journal of Earthquake Engineering* **8(2)**: 249–274.
- Ryan H (1994) *Ricker, Ormsby, Klauder, Butterworth – a Choice of Wavelets*. Canadian Society of Exploration Geophysicists, Calgary, AB, Canada.
- Schanz T, Vermeer PA and Bonnier PG (1999) The hardening soil model: formulation and verification. In *Beyond 2000 in Computation Geotechnics* (Brinkgreve RBJ (ed.)). Balkema, Rotterdam, the Netherlands, pp. 281–290.
- Tsang HH, Lo SH, Xu X and Neaz Sheikh M (2012) Seismic isolation for low-to-medium-rise buildings using granulated rubber-soil mixtures: numerical study. *Earthquake Engineering and Structural Dynamics* **41(14)**: 2009–2024.

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