Pushover analysis of structures subjected to combined actions of earthquake and wind

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Abstract

This paper puts forward a framework to investigate the structural performance of buildings subject to the combined action of earthquakes and wind. The study uses a pushover analysis to determine strength capacities of low- and medium-rise concrete structures to compare versus their strength, as inferred from the Chinese code for seismic design. The analysis addresses soil-structure interactions through the scrutiny of fixed-base and flexible-base conditions of three soil types ranging between dense and soft. The parametric study covers micro meteorological winds whose mean value spans between 0.5m/s and 20m/s. The simulated multi-load scenarios induce ductility levels going from 1 to 6 to totalise 288 case studies. The proposed framework reveals that consideration of earthquake and wind simultaneous effects could modify performance levels used for design through enhancing ductility demands. This evidences that, under current design recommendations, structures located in earthquake prone areas susceptible to unexpected levels of damage.

Keywords Pushover analysis; Earthquake and wind actions; Performance-based earthquake engineering; Strength reduction factor; Soil-structure interaction

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Highlights

- (1) A framework to investigate the structural vulnerability of buildings subject to the combined action of earthquakes and wind.
- (2) Increasing awareness of the fact that multi-hazard scenarios could modify performance levels of infrastructure targeted during design, via the enhancement of ductility demands.
- (3) Evidence that, under current design recommendations, structures located in earthquake prone areas are susceptible to experience unexpected levels of damage.
- (4) Identification of new research avenues such as considering how strength reduction factors vary with local wind regimes and their integration towards more robust earthquake resisting design procedures.

Pushover analysis of structures subjected to combined actions of earthquake and wind

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This paper puts forward a framework to investigate the structural performance of buildings subject to the combined action of earthquakes and wind. The study uses a pushover analysis to determine strength capacities of low- and medium-rise concrete structures to compare versus their strength, as inferred from the Chinese code for seismic design. The analysis addresses soil-structure interactions through the scrutiny of fixed-base and flexible-base conditions of three soil types ranging between dense and soft. The parametric study covers micro meteorological winds whose mean value spans between 0.5m/s and 20m/s. The simulated multi-load scenarios induce ductility levels going from 1 to 6 to totalise 288 case studies. The proposed framework reveals that consideration of earthquake and wind simultaneous effects could modify performance levels used for design through enhancing ductility demands. This evidences that, under current design recommendations, structures located in earthquake prone areas susceptible to unexpected levels of damage.

Keywords

Pushover analysis; Earthquake and wind actions; Performance-based earthquake engineering; Strength reduction factor; Soil-structure interaction

1. Introduction

Modal pushover or response history analysis underpin performance-based earthquake engineering (PBEE) [1] design of structures as far as these enable implementing various levels of seismic excitation. The pushover analysis is a nonlinear static procedure that helps simulating structural deterioration with the progression of load, yet its simplicity tends to overcome the complex nonlinear time-history analysis [2-4]. The method also informs separate procedures to determine strength reduction factors (SRFs) that modify linear-elastic spectra for design purposes. The reduction of design forces is a key element of performance-based design, hence is adopted by major codes around the world, see for example the ASCE/SEI 7 (ASCE, 2010), FEMA P695 (ATC, 2009), Eurocode 8 (CEN, 2011) and the National Building Code of Canada (NRCC, 2015) [5-8].

Current engineering practice ignores the potential clash of major earthquake events with low to moderate winds. This is partly due to having large return periods associated to joint events compared to the life span of infrastructure. However, the fact that strong earthquakes commonly precede moderate to strong aftershocks, and that wind is constantly flowing often with low to medium intensity, but occasionally reaching high speeds, suggest that unexpected load scenarios can emerge [9]. The evidence of expanded earthquake events that could coincide with wind actions is compelling. For example, the 2011 Mw9.0 Tohoku earthquake that occurred in Japan was followed by more than 260 aftershocks of at least magnitude 5 within only one week [10]. The 2008

Ms8.0 Wenchuan earthquake that hit the Sichuan Province in China was followed by more than 600 aftershocks with magnitudes above Ms4.0 over the next 4 months [11] while the 2017 Puebla earthquake with a magnitude of Mw7.1 recorded in Mexico nearly clashed with the hurricane Katia category 2 which dissipated about one week earlier [12]. If we add this to the fact that wind is constantly flowing with a minimum speed that is equal to the averaged local wind velocity - whose probability of occurrence is of 1, and with fluctuations (peaks) whose intensity vary depending on their frequency of occurrence, then the current design assumptions concerning the unrelated action of wind and earthquakes seem questionable.

The Commonwealth Advisory Aeronautical Research Council (CAARC) developed a benchmark building model for the comparison of simulated natural winds in wind tunnels. The results from five laboratories around the globe revealed that, winds associated to return periods of 50, 10, and 1 year, tend to increase the effects of light to moderate seismic events whose spectral acceleration exceeds 0.25g in about 30%, 20%, and 10%, respectively [13]. Noting that, scenarios created by earthquakes of a wider range of magnitudes combined with aero-elastic effects induced by wind remain unknown. Duthinh and Simiu [14] concluded that structures in regions of significant wind and seismic hazards could have twice the risk of exceedance of limit than regions with only one significant hazard. Other studies have examined the structural response of buildings for combinations of seismic and wind loadings to better understand the relevant load-response mechanisms [15, 16]. Díaz and Suárez [17] indicated that within the seismic regions, the earthquake loads in combination with steady wind loads would control the wind turbine design. Mardfekri and Gardoni [18] confirmed that annual probability of failure considering seismic and hurricane hazards of wind turbines are higher than target safety level recommended by guidelines directed to industrial practice. Zhao and Maisser [19] used a multibody model to investigate the seismic behaviour of wind turbine towers in the time domain incorporating soil-structure interaction, to conclude that the wind loading should feed into the design of structures located in earthquake prone areas.

In this paper, the evaluation of joint effect of earthquakes and wind takes place by resorting pushover to non-linear static results. This involves the comparison of structural strength capacity and demand at specific performance levels. The three-dimensional finite element method enabled to formulate the mass and stiffness matrices for buildings and to quantify the influence of soil-structure interactions (SSI). Results generated in this way indicate that the method proposed could guide other studies to assessing the building seismic vulnerability to combined actions, more accurately.

2. Description of structural systems

This study considers two reinforced-concrete (RC) structures with 5 and 8 stories, respectively. These are framed with typical beam-column joints without shear walls, and lie in regions with high-seismicity. The buildings were designed for earthquake resistance according to the “Code for Seismic Design of Buildings” [20]. Both buildings cover a surface of 16m by 20m in plan (Fig. 1(a)) and have a standardised floor-to-floor height of 3m. The cross section of beams and columns for either structure are 0.5m × 0.2m and 0.5m × 0.5m, respectively, while slabs have a thickness of 0.15m. The axial compressive strength f_{ck} of all members is 20MPa, which is corresponding to the concrete grade “C30”. The yield and ultimate strengths of longitudinal and transverse rebar were set to be 335MPa and 455MPa, respectively. The dead load considered in the analysis included self-weight of structural members, additional 1.7kN/m² due to floor finishes, and 8.5kN/m acting along each beam to represent light ductile partitions and external facades. The live load was set as 2.0kN/m². All loads

are in agreement with the Code provisions. In this study, three-dimensional models of framed structures were modelled for the pushover analysis in SAP2000 version 20 [21]. The structural design addressed material properties and load configurations that are relevant to seismic-resisting structures. Beams and columns performed as nonlinear frame elements with lumped plasticity, hence could develop plastic hinges at both ends [4]. We used the default-hinge properties provided by the program and assign the PMM hinges for columns and M3 hinges for beams as recommended.

Most design codes characterise soil types based on the shear wave velocity C_s . This study investigates three types of soil, as classified by the International Building Code (IBC) [22]. These are types A, B and C, whose basic properties appear in Table 1. Impedance functions associated with rigid massless foundations controlled the simulated soil-structure interaction whereas the soil physical properties worked as a lumped parameter system at foundation level through their parameterisation with stiffness coefficients [23]. The stiffness of the idealised springs representing the soil-flexibility are highly sensitive to the size of their footing. Therefore, their dimensions either reflect the ultimate or serviceability limit state, as specified in the relevant Code of Practice [20]. From this derives that the footings underlying each column have dimensions of $4\text{m} \times 4\text{m}$, and that satisfies all the requirements.

Fig. 1. (a) Plan view of the two structures studied herein; (b) Idealisation of foundation system: K_x , K_y , K_z , K_{rx} , K_{ry} , K_{rz} are stiffnesses of equivalent soil springs along the translational and rotational degrees of freedom about X, Y and Z axes, respectively.

Table 1 Properties of the three considered soil types adopted in IBC [22]

Soil type	Soil profile	Range of shear wave velocity, C_s	Average shear wave velocity (m/s)	Poisson's ratio, ν	Initial shear modulus, G_0 (kN/m^2) $\times 10^3$	Effective shear modulus $G = 0.42G_0$ (kN/m^2) $\times 10^3$
Very dense						
A	soil and soft rock	$360 < C_s < 750$	564	0.25	67.5	28.4
B	Stiff soil	$180 < C_s < 360$	270	0.25	13.7	5.75
C	Soft soil	$C_s < 180$	160	0.25	4.4	1.85

The three translational springs located at the base of the ground floor columns, two in principal horizontal and

one in the vertical direction, together with rotational springs about these mutually perpendicular axes, simulate soil-flexibility (Fig. 1(b)). The boundary element method and experimental tests [24-25] were in use to calibrate the properties of springs as these were hypothetically resting on homogeneous elastic half-space. The modelling technique, mapped from Gazetas [26] and reflected in the set of equations listed in Table 2, is shared practice to dealing with soil-structure interaction - see for example [23-25, 27] hence considered suitable for the purpose of this investigation. Its main features account dependability of spring stiffness on mechanical characteristics of the soil material that supports the structure as well as on the dimensions of the foundation. In addition, the mechanical characteristics of the foundation soil medium translate into an effective shear modulus G and Poisson's ratio ν . Here the effective shear modulus G relates to the initial shear modulus G_0 .

Although the foundation embedment depth did not influence the estimation of stiffness for springs, it did feed in to computing the bearing capacity of the simulated soil-flexibility. According to Bowles [28] the depth of the foundation has little influence on the bearing capacity of springs, hence its consideration seems confined to cases where controlled construction conditions do exist. As pointed out before, the modelling of soil as outlined here enables scrutinising soil-structure interactions on earthquake prone areas underlain by three different soil types. This complements the analysis of fixed-base condition of the subject buildings, for comparison. **The fixed-base disregards soil-flexibilities. Bearing in mind that the flexibility of soil causes an overall decrease in lateral stiffness leading to the lengthening of lateral natural periods, we regard the base which considers the soil-flexibility as the flexible-base.** The motivation of that parametric study derived from past studies that show how SSI vary displacement and acceleration demands of structures, which contrast with results generated with the fixed-base criteria [29-30].

Table 2 Expressions for stiffnesses of equivalent springs along various degrees of freedom as presented elsewhere [26] and used in [23-25]

Degrees of freedom	Stiffness of equivalent soil spring
Vertical	$(\frac{2GL}{1-\nu})[0.73 + 1.54(\frac{A_b}{4L^2})^{0.75}]$
Horizontal (lateral direction)	$(\frac{2GL}{2-\nu})[2 + 2.5(\frac{A_b}{4L^2})^{0.85}]$
Horizontal (longitudinal direction)	$(\frac{2GL}{2-\nu})[2 + 2.5(\frac{A_b}{4L^2})^{0.85}] - (\frac{0.2}{0.75-\nu})GL[1 - (\frac{B}{L})]$
Rocking (about the longitudinal)	$(\frac{G}{1-\nu})I_{bx}^{0.75}(\frac{L}{B})^{0.25}[2.4 + 0.5(\frac{B}{L})]$
Rocking (about the lateral)	$(\frac{3G}{1-\nu})I_{by}^{0.75}(\frac{L}{B})^{0.15}$
Torsion	$3.5GI_{bz}^{0.75}(\frac{B}{L})^{0.4}(\frac{I_{bz}}{B^4})^{0.2}$

A_b -- Area of the foundation considered.

B and L -- Half-width and half-length of a rectangular foundation, respectively.

I_{bx} , I_{by} , and I_{bz} -- Moment of inertia of the foundation area with respect to longitudinal, lateral and vertical axes, respectively.

3. Capacity and demand spectra

We adopted the capacity spectrum method recommended by ATC-40 [32] to determine the performance point i.e. the point of intersection between capacity and demand spectra. In this approach, the pushover analysis relates shear force at the base and roof displacement, into a curve representing the overall structural response. The shear forces at the base and corresponding roof displacements, once transformed into spectral accelerations and spectral displacements, respectively, constitute a capacity spectrum. The demand of strength against earthquake ground motions comes from the local elastic response spectra of acceleration. There is also a transformation process here, according to which the diagram showing spectral acceleration versus period becomes the Acceleration-Displacement Response Spectrum (ADRS). It follows that the intersection of the capacity spectrum and the demand spectrum provides an estimation of the inelastic performance of the structure.

3.1 Capacity curve

The pushover analysis, as applied herein, induced a monotonically increasing displacement-controlled lateral load pattern, in the presence of constant gravity load, until an ultimate condition appeared. The applied lateral loads relate to accelerations that the structure would experience during ground shakings. Under incrementally increasing loading, some structural members may yield sequentially. Consequently, the structural stiffness changes past the yield point, denoted as B in Fig. 2. This figure also illustrates the acceptance criteria as defined by ATC-40 [32]. That criterion defines specific performance thresholds such as Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP), corresponding to 10%, 60%, and 90% of plastic hinge deformation capacity, respectively. Thus, the two investigated structures were subject to gravitational forces and the incremental lateral loading. The latter increased step-by-step in the nonlinear static analysis, keeping displacements under control via target tip node displacements of 4% with respect to the total height of the structure, as specified in ATC-40 [32]. This process included P-delta effects to defining total forces acting on structural members hence the respective deformations.

Fig. 2. Force-deformation relationship of a typical plastic mechanism

3.2 Seismic demand of multi-hazard scenarios according to STF factor

For an elastic SDOF system the following relation applies:

$$S_{de} = \frac{T^2}{4\pi^2} S_{ae} \quad (1)$$

where S_{ae} and S_{de} are values in the elastic spectrum of acceleration and displacement, respectively, corresponding to the period T and a fixed viscous damping ratio. For an inelastic SDOF system with a bilinear

force-deformation relationship, the acceleration spectrum (S_a) and displacement spectrum (S_d) can be determined as [33]:

$$S_a = \frac{S_{ae}}{R_\mu} \quad (2)$$

$$S_d = \frac{\mu}{R_\mu} S_{ae} = \frac{\mu}{R_\mu 4\pi^2} S_{ae} = \mu \frac{T^2}{4\pi^2} S_a \quad (3)$$

where μ is the ductility factor defined as the ratio between the maximum displacement and the yield displacement. R_μ is the strength reduction factor, defined as the ratio of the elastic strength demand $F_y(\mu = 1)$ to the inelastic yield strength demand $F_y(\mu_t)$ for a given target ductility ratio (μ_t), which is represented by the following equation [34]:

$$R_\mu = F_y(\mu = 1)/F_y(\mu = \mu_t) \quad (4)$$

In this study, the acceleration response spectra $S_a(T)$ was obtained from the “Code for Seismic Design of Buildings” [20], as follows:

$$\alpha = [0.45 + 10(\eta_2 - 0.45)T]\alpha_{\max} \quad 0 < T \leq 0.1s \quad (5)$$

$$\alpha = \eta_2 \alpha_{\max} \quad 0.1s < T \leq T_g \quad (6)$$

$$\alpha = (T_g/T)^\gamma \eta_2 \alpha_{\max} \quad T_g < T \leq 5T_g \quad (7)$$

$$\alpha = [0.2^\gamma \eta_2 - \eta_1(T - 5T_g)]\alpha_{\max} \quad 5T_g < T \leq 6s \quad (8)$$

where α - seismic influence coefficient; $\alpha_{\max}$ - maximum value of seismic influence coefficient; γ - attenuation index of curvilinear descending section, $\gamma = 0.9 + (0.05 - \xi)/(0.5 + 5\xi)$; ξ - damping ratio; η_1 - adjustment coefficient for the descending slope in the linear decreasing region, $\eta_1 = 0.02 + (0.05 - \xi)/8$, and if $\eta_1 < 0$, set $\eta_1 = 0$; η_2 - damping adjustment coefficient, $\eta_2 = 1 + (0.05 - \xi)/(0.06 + 1.7\xi)$; T - period of the structure; T_g - characteristic period. The curve defining the seismic influence coefficient is shown in Fig. 3.

A constant damping ratio ξ of 5% was set for the acceleration response spectra [35]. The value of the characteristic period T_g depends on the site class and design earthquake group, which were set as “category II” (the basic seismic precautionary criterion specified in the Code is the acceleration index under the site class of category II) and “group 2” (which stands for intermediate distance earthquakes). This defined the value of T_g as 0.40s. The maximum value of seismic influence coefficient $\alpha_{\max}$ was set as 0.9g, corresponding to rare earthquake occurrence in areas of intensity 8.

Fig. 3. Seismic influence coefficient curve in the Code

According to Vidic et al. and Fajfar [33, 36], R_μ is a function of structural period, target ductility, and characteristic parameters of the relevant hysteretic model. In this study, the R_μ - T model introduced by Vidic et al. [33] was adopted hence the value of R_μ was approximated with a bilinear curve:

$$R_\mu = c_1(\mu - 1)^{c_{R\frac{T}{T_0}}} + 1 \quad T \leq T_0 \quad (9)$$

$$R_\mu = c_1(\mu - 1)^{C_R} + 1 \quad T > T_0 \quad (10)$$

where T_0 is the period dividing the period range into two portions. T_0 is related to the predominant period of the ground motion T_g by means of:

$$T_0 = c_2 \mu^{C_T} T_g \quad (11)$$

The coefficients c_1 , c_2 , C_R and C_T depend on the assumed hysteretic behaviour and damping. The Q-model (mass-proportional damping) was adopted here, therefore according to it c_1 , c_2 , C_R and C_T equal to 1.0, 1.0, 0.65 and 0.30, respectively. The R_μ -T model is shown in Fig. 4(a). In this process, we regard the same strength reduction factor R_μ for the selected support conditions of the buildings namely, fixed-base and flexible-base, which obeys the SSI regulations in the current seismic design codes (ASCE [5]). In other words, it was assumed that SSI has a similar impact on the strength demand of either elastic or inelastic structures. Starting from the typical elastic design spectrum according to the Code (equations (5)-(8)), and by using equations (2), (3), (9)-(11), the corresponding demand spectra for the different ductility factors μ in ADRS format were obtained (Fig. 4(b)).

Fig. 4. (a) R_μ -T model (Q-model, mass-proportional damping) proposed by Vidic et al. [31]; (b) Demand spectra of different ductility level μ in ADRS format

According to Martinez-Vazquez [37], the effect of multiple hazard scenarios on structures whose response is dominated by the fundamental vibrational mode, can be done through generalising their distributed mass (M^*) while defining a mass to volume ratio parameter (Γ). The generalised mass (M^*) can be calculated using equation (12):

$$M^* = \int_0^H \phi(z)^2 M(z) dz \quad (12)$$

where $M(z)$ represents the structural mass per unit length, z is a vertical coordinate and ϕ is the fundamental modal shape, which was approximated by $\phi(z) = (z/H)^\beta$ where $\beta = 1.5$ and H is the height of the building. The geometries of the prototype building are shown in Fig. 5. This model essentially defines generalised MDOF systems that preserve their volume. The purpose of this is to use the model to quantify the inelastic performance of structures subject to earthquake and wind combined load. The method is analogous to standard techniques used in earthquake engineering to monitoring the inelastic performance of single oscillators leading to determine SRFs. To that end, Martinez-Vazquez defined the mass to volume ratio $\Gamma = M^* / (H^2 + W^2 + L^2)^{1/2}$ to characterising buildings. In the present investigation, Γ took values of $\Gamma_{5F} = 22.39 \text{ ton/m}$ and $\Gamma_{8F} = 28.13 \text{ ton/m}$ for the two investigated structures, respectively.

Fig. 5. Geometry of full-scale structures for analysis (Martinez-Vazquez [37])

With the available SRF database from Martinez-Vazquez while considering the averaged wind velocity $\bar{U}$ to be 0.5 m/s to 20m/s and inelastic levels μ ranging from 1 to 6, we derived the ratio R^*_μ/R_μ - where R^*_μ represents the strength reduction factor addressing multi-hazard scenarios induced by earthquake and wind loading. This required interpolating the curves given in [37] using the Lagrange Interpolation Polynomial method, with respect to the mass to volume ratio Γ_{5F} and Γ_{8F} that characterise our target buildings. The resulting values of R^*_μ/R_μ , once averaged over the ranges $T \leq 2s$ and $T > 2s$ for the 5-storey and 8-storey buildings are listed in Table 3 and Table 4, respectively. The criterion of sub-dividing the interval of $0.1s \leq T \leq 5s$ specified for SRFs in [37] reflects the fact that the original curves show different steadiness past $T=2s$. It follows that a revised demand spectra associated to different inelastic levels μ can be produced, now using the SRFs related to multi-hazards conditions. Fig. 6 shows the deduced R^*_μ -T model for the 5-storey building for when $0.5m/s \leq \bar{U} \leq 20m/s$. In addition, Fig. 7 compares the original and revised demand spectra for different inelastic level μ for the 5-storey building subject to a wind velocity of $\bar{U}=20m/s$.

Table 3 Derived R^*_μ/R_μ with $\bar{U}$, Γ and μ averaged across $T \leq 2s$

		μ					
	$\bar{U}$: m/s	1	2	3	4	5	6
$\Gamma_{5F}=22.39$	0.5	1.000	0.994	0.969	0.944	0.912	0.917
	5	1.000	0.951	0.935	0.894	0.872	0.836
	10	1.000	0.905	0.852	0.787	0.753	0.721
	15	1.000	0.871	0.783	0.707	0.672	0.633
	20	1.000	0.850	0.766	0.674	0.618	0.562
$\Gamma_{8F}=28.13$	0.5	1.000	0.991	0.964	0.935	0.898	0.904
	5	1.000	0.956	0.947	0.911	0.892	0.858
	10	1.000	0.919	0.871	0.821	0.787	0.752
	15	1.000	0.893	0.812	0.736	0.704	0.663
	20	1.000	0.872	0.795	0.703	0.647	0.589

Table 4 Derived R^*_μ/R_μ with $\bar{U}$, Γ and μ averaged across $T > 2s$

		μ					
	$\bar{U}$: m/s	1	2	3	4	5	6
$\Gamma_{5F}=22.39$	0.5	0.998	1.002	0.984	0.978	0.947	0.958
	5	0.995	0.957	0.862	0.782	0.711	0.662
	10	0.995	0.784	0.636	0.530	0.454	0.400
	15	0.996	0.732	0.552	0.436	0.358	0.306

	20	0.997	0.683	0.501	0.392	0.320	0.272
$\Gamma_{8F}=28.13$	0.5	0.998	1.004	0.987	0.983	0.950	0.964
	5	0.996	0.977	0.890	0.814	0.743	0.695
	10	0.995	0.804	0.662	0.557	0.481	0.426
	15	0.996	0.742	0.564	0.448	0.369	0.317
	20	0.998	0.693	0.512	0.402	0.329	0.280

Fig. 6. Deduced R^*_μ -T model and corresponding revised demand spectra of different inelastic level μ for the 5-storey building. (a) $\bar{U} = 0.5\text{m/s}$; (b) $\bar{U} = 5\text{m/s}$; (c) $\bar{U} = 10\text{m/s}$; (d) $\bar{U} = 15\text{m/s}$; (e) $\bar{U} = 20\text{m/s}$

Fig. 7. Comparison of the original and revised demand spectra of different inelastic level μ for the 5-storey building ($\bar{U}=20\text{m/s}$)

Following the procedure detailed above, we deduced the R^*_μ -T model for the 8-storey building reaching different inelastic levels μ and subject to different wind velocities $\bar{U}$. The results showed similar trends than those illustrated in Fig. 6, hence those not presented here for brevity. In general, the results obtained show that the amplitude and shape of the R^*_μ -T curves depend on the wind velocity $\bar{U}$ and the ratio of mass to volume Γ . The observed decrease of R^*_μ/R_μ fluctuated between 20-60%, including low wind level scenarios, but go below these limits with increased average wind speed.

4 Pushover analysis results

The results presented above enable comparing strength demands for the two buildings, associated to various ductility levels. Based on the pushover analysis performed with SAP2000, a capacity curve of the structure was generated (roof displacement verses the base shear) for each load case. For the sake of clarity, the identified intersection between the capacity and demand curves appear in Table 5 and Table 6. These Tables primarily

show that for all systems to maintaining target ductility levels the displacement and force demands increase gradually with the wind velocity. Evidently, strength demands in any form reduce inversely proportional to the ductility μ .

The capacity curves shown in Fig. 8 and Fig. 9 illustrate performance levels for investigated different wind speeds $\bar{U}$ and support conditions, related to the 5-storey and 8-storey buildings, respectively. The solid line stands for the capacity curve derived from the fixed-base condition while the dashed line stands for the capacity derived from using soil type C; the discrete symbols correspond to performance points: circles are for $\mu=1$, squares are for $\mu=3$ and triangles are for $\mu=6$; black markers represent the action of single earthquakes and the red ones represent the combined action of earthquakes and wind. Two curves are included in these figures to represent the fixed-base case and most flexible-base case, correspondingly. The ordinates of the estimated performance point stress the fact that the higher the wind velocity or inelastic level is, the larger the difference between the strength demands associated to earthquake and wind joint events and earthquake forces acting alone.

Table 5 Top displacements, D and base shear, V at the performance point for the 5-storey building

$\bar{U}$: m/s		Fixed-base		Soil A		Soil B		Soil C	
		D(m)	V(kN)	D(m)	V(kN)	D(m)	V(kN)	D(m)	V(kN)
0	$R_{\mu=1}$	0.054	3090.559	0.057	3102.572	0.066	3194.387	0.095	3494.404
	$R_{\mu=2}$	0.024	2267.470	0.026	2262.466	0.032	2233.070	0.046	1944.033
	$R_{\mu=3}$	0.016	1739.251	0.017	1752.036	0.022	1704.224	0.029	1234.439
	$R_{\mu=4}$	0.012	1391.398	0.013	1368.410	0.016	1274.779	0.021	905.610
	$R_{\mu=5}$	0.010	1179.780	0.010	1137.715	0.013	1024.876	0.017	716.414
	$R_{\mu=6}$	0.008	1037.948	0.009	984.251	0.010	861.292	0.014	592.395
0.5	$R_{\mu=1}^*$	0.054	3086.243	0.057	3098.634	0.066	3188.582	0.095	3491.417
	$R_{\mu=2}^*$	0.025	2276.577	0.026	2271.487	0.032	2241.933	0.046	1958.826
	$R_{\mu=3}^*$	0.017	1783.110	0.018	1800.796	0.022	1759.873	0.030	1276.883
	$R_{\mu=4}^*$	0.013	1453.737	0.014	1436.726	0.017	1350.119	0.023	963.217
	$R_{\mu=5}^*$	0.011	1262.125	0.011	1227.207	0.014	1121.114	0.018	788.747
	$R_{\mu=6}^*$	0.009	1102.339	0.010	1053.809	0.011	935.202	0.015	648.392
5	$R_{\mu=1}^*$	0.054	3086.243	0.057	3098.634	0.066	3188.582	0.095	3491.417
	$R_{\mu=2}^*$	0.026	2339.676	0.027	2334.073	0.034	2303.291	0.049	2060.282
	$R_{\mu=3}^*$	0.017	1835.892	0.019	1859.622	0.023	1827.645	0.031	1329.347
	$R_{\mu=4}^*$	0.014	1515.757	0.015	1504.943	0.018	1426.003	0.024	1020.802
	$R_{\mu=5}^*$	0.011	1303.985	0.012	1272.854	0.015	1170.671	0.019	826.392
	$R_{\mu=6}^*$	0.010	1176.687	0.010	1134.407	0.013	1021.206	0.017	713.206
10	$R_{\mu=1}^*$	0.054	3086.243	0.057	3098.634	0.066	3188.582	0.095	3491.417
	$R_{\mu=2}^*$	0.027	2415.752	0.029	2409.212	0.035	2376.495	0.051	2184.237
	$R_{\mu=3}^*$	0.019	1965.094	0.020	1959.085	0.025	1931.363	0.035	1472.796
	$R_{\mu=4}^*$	0.015	1674.368	0.017	1680.040	0.020	1622.198	0.028	1171.243
	$R_{\mu=5}^*$	0.013	1456.962	0.014	1440.347	0.017	1354.216	0.023	966.001
	$R_{\mu=6}^*$	0.011	1311.794	0.012	1281.415	0.015	1179.772	0.020	833.316
15	$R_{\mu=1}^*$	0.054	3086.243	0.057	3098.634	0.066	3188.582	0.095	3491.417
	$R_{\mu=2}^*$	0.028	2465.851	0.030	2458.479	0.036	2424.277	0.053	2266.510

	$R_{\mu=3}^*$	0.021	2060.303	0.022	2055.188	0.028	2027.634	0.038	1618.539
	$R_{\mu=4}^*$	0.017	1824.098	0.018	1846.463	0.023	1812.372	0.031	1317.453
	$R_{\mu=5}^*$	0.015	1590.096	0.016	1586.878	0.019	1517.45	0.026	1090.927
	$R_{\mu=6}^*$	0.013	1448.063	0.014	1430.449	0.017	1342.927	0.022	957.581
20	$R_{\mu=1}^*$	0.054	3086.243	0.057	3098.634	0.066	3188.582	0.095	3491.417
	$R_{\mu=2}^*$	0.029	2502.224	0.031	2509.887	0.037	2473.979	0.055	2352.928
	$R_{\mu=3}^*$	0.021	2108.201	0.023	2082.443	0.028	2054.816	0.039	1660.445
	$R_{\mu=4}^*$	0.018	1894.147	0.019	1902.254	0.024	1873.764	0.033	1387.916
	$R_{\mu=5}^*$	0.016	1699.211	0.017	1707.536	0.021	1653.169	0.028	1195.125
	$R_{\mu=6}^*$	0.014	1585.641	0.015	1581.901	0.019	1511.706	0.026	1086.664

Fig. 8. Capacity curves and performance points associated to different inelastic levels μ and wind speeds $\bar{U}$ for the 5-storey building.

Table 6 Top displacements, D and base shear, V at the performance point for the 8-storey building

$\bar{U}$: m/s		Fixed-base		Soil A		Soil B		Soil C	
		D(m)	V(kN)	D(m)	V(kN)	D(m)	V(kN)	D(m)	V(kN)
0	$R_{\mu=1}$	0.095	3210.033	0.100	3231.834	0.116	3281.823	0.172	3611.394
	$R_{\mu=2}$	0.042	2110.619	0.046	2111.006	0.057	2148.579	0.080	1845.563
	$R_{\mu=3}$	0.027	1639.075	0.030	1638.572	0.039	1691.939	0.050	1170.261
	$R_{\mu=4}$	0.020	1416.417	0.022	1411.233	0.028	1299.775	0.037	858.573
	$R_{\mu=5}$	0.016	1189.947	0.018	1169.941	0.022	1027.504	0.029	679.379
	$R_{\mu=6}$	0.014	1029.361	0.015	994.200	0.018	851.582	0.024	563.449
0.5	$R^*_{\mu=1}$	0.095	3210.033	0.100	3231.834	0.116	3281.823	0.172	3611.394
	$R^*_{\mu=2}$	0.043	2123.612	0.046	2123.846	0.058	2160.453	0.081	1864.604

	$R^*_{\mu=3}$	0.028	1672.605	0.031	1672.543	0.040	1726.098	0.053	1217.908
	$R^*_{\mu=4}$	0.022	1462.204	0.024	1458.296	0.031	1397.876	0.040	922.095
	$R^*_{\mu=5}$	0.018	1297.040	0.020	1288.595	0.025	1150.051	0.033	760.020
	$R^*_{\mu=6}$	0.015	1114.872	0.016	1087.273	0.020	943.999	0.027	624.634
5	$R^*_{\mu=1}$	0.095	3210.036	0.100	3231.834	0.116	3281.823	0.172	3611.394
	$R^*_{\mu=2}$	0.045	2178.293	0.048	2177.767	0.060	2210.315	0.084	1944.094
	$R^*_{\mu=3}$	0.029	1689.593	0.031	1689.714	0.041	1743.001	0.054	1241.578
	$R^*_{\mu=4}$	0.022	1480.931	0.024	1477.446	0.032	1438.151	0.041	948.473
	$R^*_{\mu=5}$	0.018	1303.902	0.020	1296.169	0.025	1157.328	0.033	765.337
	$R^*_{\mu=6}$	0.016	1162.088	0.017	1139.261	0.022	996.131	0.028	658.918
10	$R^*_{\mu=1}$	0.095	3210.036	0.100	3231.834	0.116	3281.823	0.172	3611.394
	$R^*_{\mu=2}$	0.047	2241.765	0.050	2240.241	0.062	2267.741	0.088	2037.178
	$R^*_{\mu=3}$	0.032	1774.964	0.034	1775.973	0.044	1828.123	0.059	1362.320
	$R^*_{\mu=4}$	0.025	1561.086	0.027	1559.307	0.035	1610.328	0.046	1060.759
	$R^*_{\mu=5}$	0.021	1427.257	0.023	1422.416	0.029	1323.445	0.038	873.788
	$R^*_{\mu=6}$	0.018	1292.061	0.020	1283.056	0.025	1143.560	0.032	756.547
15	$R^*_{\mu=1}$	0.095	3210.036	0.100	3231.834	0.116	3281.823	0.172	3611.394
	$R^*_{\mu=2}$	0.048	2290.128	0.051	2287.756	0.064	2311.262	0.091	2108.728
	$R^*_{\mu=3}$	0.034	1852.372	0.037	1853.726	0.047	1904.000	0.064	1473.136
	$R^*_{\mu=4}$	0.028	1656.105	0.030	1655.815	0.039	1709.331	0.051	1193.908
	$R^*_{\mu=5}$	0.023	1506.798	0.025	1503.874	0.033	1493.212	0.042	984.367
	$R^*_{\mu=6}$	0.020	1420.146	0.022	1415.205	0.029	1308.366	0.037	863.657
20	$R^*_{\mu=1}$	0.095	3210.036	0.100	3231.834	0.116	3281.823	0.172	3611.394
	$R^*_{\mu=2}$	0.050	2330.047	0.053	2326.769	0.065	2346.335	0.094	2167.305
	$R^*_{\mu=3}$	0.035	1877.194	0.038	1878.592	0.048	1928.169	0.065	1508.877
	$R^*_{\mu=4}$	0.029	1699.301	0.032	1699.581	0.041	1752.897	0.054	1255.294
	$R^*_{\mu=5}$	0.025	1573.372	0.028	1571.808	0.036	1624.668	0.046	1077.605
	$R^*_{\mu=6}$	0.023	1502.966	0.025	1499.992	0.033	1485.720	0.042	979.212

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Fig. 9. Capacity curves and performance points associated to different inelastic levels μ and wind speeds $\bar{U}$ for the 8-storey building.

To draw more general conclusions, the rate of variation in base shear and roof displacement due to earthquake and wind joint effects were investigated. This involved both, the 5-storey and 8-storey buildings with consideration of soil-structure interaction. The corresponding results are shown in Tables 7 to 10, which highlight the potential impact of multi-hazard events on buildings. These results reiterate that strength and displacement demands would increase with wind speed and inelastic levels at the time that dimensionalise the change.

In light of these results, the impact of wind speed $\bar{U}$ on the value of performance points seems remarkable. The rate of increase of both force and displacement demands is considerable even for low values of $\bar{U}$. For example, the capacity demand of 5-storey building with fixed-base associated to $\bar{U}=0.5\text{m/s}$, see Table 7, shows that the force required to maintain target ductility values is of 0.4%, 2.52%, 4.48%, 6.98% and 6.2% for μ equal to 2, 3, 4, 5 and 6 respectively. Furthermore, the increased rate in displacement demand shown in Table 9 is even higher: 4.17%, 6.25%, 8.33%, 10% and 12.5% corresponding to $\mu=2-6$, respectively. Generally the highest changes would occur within the intervals of $0.5\text{m/s} \leq \bar{U} \leq 10\text{m/s}$ regardless of the structural heights and support conditions.

Table 7 Rate of change (%) in base shear for the 5-storey building related to different wind velocity intervals

	Deviation %	$\Delta\bar{U}_{0.5} - \bar{U}_0$	$\Delta\bar{U}_5 - \bar{U}_{0.5}$	$\Delta\bar{U}_{10} - \bar{U}_5$	$\Delta\bar{U}_{15} - \bar{U}_{10}$	$\Delta\bar{U}_{20} - \bar{U}_{15}$
Fixed-base	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	0.40	2.77	3.25	2.07	1.48
	$R^*_{\mu=3}$	2.52	2.96	7.04	4.85	2.32
	$R^*_{\mu=4}$	4.48	4.27	10.46	8.94	3.84
	$R^*_{\mu=5}$	6.98	3.32	11.73	9.14	6.86
	$R^*_{\mu=6}$	6.20	6.74	11.48	10.39	9.50
Soil A	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	0.40	2.76	3.22	2.04	2.09
	$R^*_{\mu=3}$	2.78	3.27	5.35	4.91	1.33
	$R^*_{\mu=4}$	4.99	4.75	11.63	9.91	3.02
	$R^*_{\mu=5}$	7.87	3.72	13.16	10.17	7.60
	$R^*_{\mu=6}$	7.07	7.65	12.96	11.63	10.59
Soil B	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00

	$R^*_{\mu=2}$	0.40	2.74	3.18	2.01	2.05
	$R^*_{\mu=3}$	3.27	3.85	5.67	4.98	1.34
	$R^*_{\mu=4}$	5.91	5.62	13.76	11.72	3.39
	$R^*_{\mu=5}$	9.39	4.42	15.68	12.05	8.94
	$R^*_{\mu=6}$	8.58	9.20	15.53	13.83	12.57
Soil C	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	0.76	5.18	6.02	3.77	3.81
	$R^*_{\mu=3}$	3.44	4.11	10.79	9.90	2.59
	$R^*_{\mu=4}$	6.36	5.98	14.74	12.48	5.35
	$R^*_{\mu=5}$	10.10	4.77	16.89	12.93	9.55
	$R^*_{\mu=6}$	9.45	10.00	16.84	14.91	13.48

Table 8 Rate of change (%) in base shear for the 8-storey building related to different wind velocity intervals

	Deviation %	$\Delta \bar{U}_{0.5} - \bar{U}_0$	$\Delta \bar{U}_5 - \bar{U}_{0.5}$	$\Delta \bar{U}_{10} - \bar{U}_5$	$\Delta \bar{U}_{15} - \bar{U}_{10}$	$\Delta \bar{U}_{20} - \bar{U}_{15}$
Fixed-base	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	0.62	2.57	2.91	2.16	1.74
	$R^*_{\mu=3}$	2.05	1.02	5.05	4.36	1.34
	$R^*_{\mu=4}$	3.23	1.28	5.41	6.09	2.61
	$R^*_{\mu=5}$	9.00	0.53	9.46	5.57	4.42
	$R^*_{\mu=6}$	8.31	4.24	11.18	9.91	5.83
Soil A	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	0.61	2.54	2.87	2.12	1.71
	$R^*_{\mu=3}$	2.07	1.03	5.10	4.38	1.34
	$R^*_{\mu=4}$	3.33	1.31	5.54	6.19	2.64
	$R^*_{\mu=5}$	10.14	0.59	9.74	5.73	4.52
	$R^*_{\mu=6}$	9.36	4.78	12.62	10.30	5.99
Soil B	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	0.55	2.31	2.60	1.92	1.52
	$R^*_{\mu=3}$	2.02	0.98	4.88	4.15	1.27
	$R^*_{\mu=4}$	7.55	2.88	11.97	6.15	2.55
	$R^*_{\mu=5}$	11.93	0.63	14.35	12.83	8.80
	$R^*_{\mu=6}$	10.85	5.52	14.80	14.41	13.56
Soil C	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	1.03	4.26	4.79	3.51	2.78
	$R^*_{\mu=3}$	4.07	1.94	9.72	8.13	2.43
	$R^*_{\mu=4}$	7.40	2.86	11.84	12.55	5.14
	$R^*_{\mu=5}$	11.87	0.70	14.17	12.66	9.47
	$R^*_{\mu=6}$	10.86	5.49	14.82	14.16	13.38

Table 9 Rate of change (%) in roof displacement for the 5-storey building related to different wind velocity intervals

	Deviation %	$\Delta \bar{U}_{0.5} - \bar{U}_0$	$\Delta \bar{U}_5 - \bar{U}_{0.5}$	$\Delta \bar{U}_{10} - \bar{U}_5$	$\Delta \bar{U}_{15} - \bar{U}_{10}$	$\Delta \bar{U}_{20} - \bar{U}_{15}$
Fixed-	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00

base	$R^*_{\mu=2}$	4.17	4.00	3.85	3.70	3.57
	$R^*_{\mu=3}$	6.25	0.00	11.76	10.53	0.00
	$R^*_{\mu=4}$	8.33	7.69	7.14	13.33	5.88
	$R^*_{\mu=5}$	10.00	0.00	18.18	15.38	6.67
	$R^*_{\mu=6}$	12.50	11.11	10.00	18.18	7.69
Soil A	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	0.00	3.85	7.41	3.45	3.33
	$R^*_{\mu=3}$	5.88	5.56	5.26	10.00	4.55
	$R^*_{\mu=4}$	7.69	7.14	13.33	5.88	5.56
	$R^*_{\mu=5}$	10.00	9.09	16.67	14.29	6.25
	$R^*_{\mu=6}$	11.11	0.00	20.00	16.67	7.14
Soil B	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	0.00	6.25	2.94	2.86	2.78
	$R^*_{\mu=3}$	0.00	4.55	8.70	12.00	0.00
	$R^*_{\mu=4}$	6.25	5.88	11.11	15.00	4.35
	$R^*_{\mu=5}$	7.69	7.14	13.33	11.76	10.53
	$R^*_{\mu=6}$	10.00	18.18	15.38	13.33	11.76
Soil C	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	0.00	6.52	4.08	3.92	3.77
	$R^*_{\mu=3}$	3.45	3.33	12.90	8.57	2.63
	$R^*_{\mu=4}$	9.52	4.35	16.67	10.71	6.45
	$R^*_{\mu=5}$	5.88	5.56	21.05	13.04	7.69
	$R^*_{\mu=6}$	7.14	13.33	17.65	10.00	18.18

Table 10 Rate of change (%) in roof displacement for the 8-storey building related to different wind velocity intervals

	Deviation %	$\Delta \bar{U}_{0.5} - \bar{U}_0$	$\Delta \bar{U}_5 - \bar{U}_{0.5}$	$\Delta \bar{U}_{10} - \bar{U}_5$	$\Delta \bar{U}_{15} - \bar{U}_{10}$	$\Delta \bar{U}_{20} - \bar{U}_{15}$
Fixed-base	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	2.38	4.65	4.44	2.13	4.17
	$R^*_{\mu=3}$	3.70	3.57	10.34	6.25	2.94
	$R^*_{\mu=4}$	10.00	0.00	13.64	12.00	3.57
	$R^*_{\mu=5}$	12.50	0.00	16.67	9.52	8.70
	$R^*_{\mu=6}$	7.14	6.67	12.50	11.11	15.00
Soil A	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	0.00	4.35	4.17	2.00	3.92
	$R^*_{\mu=3}$	3.33	0.00	9.68	8.82	2.70
	$R^*_{\mu=4}$	9.09	0.00	12.50	11.11	6.67
	$R^*_{\mu=5}$	11.11	0.00	15.00	8.70	12.00
	$R^*_{\mu=6}$	6.67	6.25	17.65	10.00	13.64
Soil B	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	1.75	3.45	3.33	3.23	1.56
	$R^*_{\mu=3}$	2.56	2.50	7.32	6.82	2.13
	$R^*_{\mu=4}$	10.71	3.23	9.38	11.43	5.13
	$R^*_{\mu=5}$	13.64	0.00	16.00	13.79	9.09

	$R^*_{\mu=6}$	11.11	10.00	13.64	16.00	13.79
Soil C	$R^*_{\mu=1}$	0.00	0.00	0.00	0.00	0.00
	$R^*_{\mu=2}$	1.25	3.70	4.76	3.41	3.30
	$R^*_{\mu=3}$	6.00	1.89	9.26	8.47	1.56
	$R^*_{\mu=4}$	8.11	2.50	12.20	10.87	5.88
	$R^*_{\mu=5}$	13.79	0.00	15.15	10.53	9.52
	$R^*_{\mu=6}$	12.50	3.70	14.29	15.63	13.51

To better visualise the above results, Fig.10 and fig.11 were plotted based on Table 7 to 8 and Table 9 to 10, respectively. With respect to force capacity demands derived from analysing multi-hazard conditions, Fig. 10 shows that the rate of change in the base shear due to the increased wind velocity is higher for systems with a more pronounced soil-structure interaction. As for the 5-storey building, for instance, the ratio of Rate of change (%) in force capacity demand between the the most flexible-base and the fixed-base structure for each ductility level was calculated, which gives an averaged ratio as approximately 1.55. Specifically, when the wind velocity reaches $\bar{U}=20\text{m/s}$ and the inelastic level μ is at the highest value of 6, the rate of change associate to the force capacity demand for the fixed-base subject to multi-hazard conditions is about 45% higher than that under single earthquake activity. The same parameter exhibits a gradually trend of increase with decreasing hardness of soil. That increase is roughly 50%, 60% and 70% for soil type A, B and C, respectively. The increase of force capacity demand for the 8-storey building is similar, although, the magnification of the overall rate of change is slightly lower, accounting for about 0.85 to 0.9 of the 5-storey building, for each support condition. The results presented in the form of percentage of the rate of change increase in base shear due to the soil-flexibility highlights the significance of considering this soil-structure interaction in structural analysis.

As for roof displacement capacity demands, the results indicate that multi-hazard load scenarios can also have significant impact. A comparison against equivalent rate of change in force capacity demands, show an increase of displacement demands for both buildings that could reach up to 60% when $\mu=6$ and $\bar{U}=20\text{m/s}$, for all soil types and including the fixed-base condition. It is to note however, that there is not substantial difference of such magnification of capacity demand between the fixed-base and flexible-base support conditions. This trend is also observed in Fig. 11, related to floor displacement demand for both the 5-storey and 8-storey building. In that case, the increase in the rate of change for flexible supports is of around 60-70% in the worst possible scenario. In general, the results obtained show little variation for the low- and medium-rise buildings. A potential explanation of this relates to the observed proportionality between the ratio R^*_{μ}/R_{μ} and the structural mass to volume ratio Γ , i.e. the higher the value of Γ the larger the SRF associated to multi-hazard conditions R^*_{μ} . In this investigation we obtained $\Gamma_{5F} = 22.39\text{ton/m}$, $\Gamma_{8F} = 28.13\text{ton/m}$, which implies that the 8-storey building would have a higher SRF R^*_{μ} than the 5-storey building. This derived in a lower demand of strength to withstand earthquake and wind joint load with respect to that associated to the single earthquake condition that would balance the relative differences between capacity and demand for the two case studies. The observed proportionality between R^*_{μ} and Γ also suggests that those structures with relatively low Γ are more susceptible to exceed target performance demand levels than relatively large (massive) structures if these were designed with SRFs corresponding to the zero wind load condition.

(a) (b) (c) (d)

(e) (f) (g) (h)

Fig. 10. Rate of change (%) in base shear related to wind velocity intervals. (a)-(d) results of the 5-storey and (e)-(h) results of the 8-storey building with fixed-base and soil types A to C

(a) (b) (c) (d)

(e) (f) (g) (h)

Fig. 11. Rate of change (%) in displacement related to different wind velocity intervals. (a)-(d) results of the 5-storey and (e)-(h) results of the 8-storey building with fixed-base and soil types A to C

5 Discussion and conclusions

The paper puts forward a framework to investigate structural performance under the combined action of earthquakes and wind. The procedure involves pushover analysis to determine structural strength capacities that compare against the corresponding demand. Two reinforcement concrete structures with 5 and 8 stories were investigated considering fixed-base and flexible-base conditions using three-dimensional finite element method. The latter involving three soil types ranging between dense and soft, hence addressing the impact of soil-structure interaction. The averaged wind velocity $\bar{U}$ varied between 0.5m/s and 20m/s while target ductility levels μ varied between 1 and 6, to totalise 288 case studies. The investigation revealed significant variability of structural performance due to the earthquake and wind joint effects, with respect to current design approaches solely based on earthquake loading. The proposed method could inform further studies on seismic vulnerability for structures subject to multi-hazard scenarios.

The current seismic design codes recommend the same strength reduction factors for both fixed-base and flexible-base structures (eg, ASCE [5]). This consideration implicitly assume that the reduction of design base shear for both elastic and inelastic building performance is proportional. However, the results obtained suggest that of the impact of SSI on inelastic performance is not as clear as it is for elastic performance. In other words, the lack of distinction amongst the referred support conditions, as reflected in the code recommendations, would cause excessive reduction of the yield strength for the flexible-base structures, and in turn, lead to higher displacement demands in the flexible-base structure compared with the fixed-base structure. These findings are illustrated in Table 5 and Table 6, which also show that displacement demands in flexible-base structures exceed those of fixed-base structures analysed by using the same strength reduction factor (while of the opposite is true when looking at results involving force demands). Furthermore, the rate of force demand for flexible-base structures increases with the wind speed at higher rates than it does for fixed-base structures.

The analysis of the probabilities of wind occurrence in normal conditions reported in [13] suggests that in certain regions wind velocities of 5m/s are susceptible to be exceeded in up to 75% of time, whereas those within the range of $5\text{m/s} < \bar{U} < 10\text{m/s}$ can be exceeded 25% of time. It therefore seems non-conservative to ignore the potential occurrence of earthquake and wind joint events during the design stage. The criteria reflected in current codes of practice assume that return periods associated to joint events exceed those related to single load action, however such considerations ignore that the return period of ground motions is not fixed and can vary significantly from one area to another [38]. In NOAA [39] is shown that the number of earthquakes with magnitude above 6.5 recorded in the last 50 years totalise 789, whereas 118 and 18 of those events have occurred within the last 5 years and during 2018, respectively. This combined with the number of recorded aftershocks that follow major earthquake events make infrastructure susceptible to multi-hazard scenarios induced by earthquakes and wind, both during construction and once in operation. This is a permanent risk that to date remains unquantified. The proposed framework recognises that integrating wind actions on earthquake resisting design procedures would enable mitigating such risk, particularly when such approach takes into account seismic regionalisation. In that context, it is recommended that an integrated risk analysis considers the variation of wind regimes across different areas, once it has been found that SRFs are highly dependent on the relative magnitude of earthquake and wind forces. This measure could be more effective than modifying load combination factors for wind resisting design when seismic load counts as additional variable load.

The proposed method was verified with the conventional and typical low-rise and medium-rise buildings by using the same foundation type, hence addressing SSI effects across different case studies. We recognise the fact that high-rise buildings take higher risks under wind actions due their aero elasticity, thus a future prospect would investigate the effect of multi-hazard scenarios on foundation types that are proper of high-rise buildings.

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