A GROUND MOTION MODEL IN PROXIMITY OF VIBRATING BUILDINGS

P. CACCIOLA AND A.TOMBARI

School of Environment and Technology, University of Brighton, Cockcroft Building Lewes Road BN2 4GJ, Brighton, UK.

Ground motion in urban environment is modified by the presence of buildings, mainly this is due to the radiation energy emitted from a vibrating structure in the soil that alters the seismic free field motion. This effect is part of the bigger phenomenon referred to as site-city-interaction. Furthermore, ground response modifications induced by the site-city-interaction effect inside the city may also contaminate the ground motion outside the city because of the wave field radiated from the buildings. Traditional stochastic ground motion models used for the seismic design of structures and infrastructures take into account the soil deposit only, disregarding the presence of existing buildings nearby. This study is a first attempt to propose a ground motion analytical model able to take into account the influence of the urban environment. A simplified discrete model is developed so to consider its influence on the free field ground motion. Comparison in terms power spectral density functions and peak ground acceleration determined from the proposed ground motion model and those derived by conventional approaches are carried out. Numerical results will show the potentiality of the proposed approach to capture this complex phenomenon in the design process by improving the accuracy of the estimation of the response of structures to ground motion in the proximity of another building of above 30%.

Keywords: Seismic site-city interaction, ground motion model, stochastic response.

1. Introduction

Modelling of the earthquake induced ground motion is a still open public safety issue that need to be addressed to better predict the probability of failure of structure and infrastructure and to protect ultimately human lives. There is nowadays no universal recognized earthquake ground motion model although progresses have been made in the last few decades toward the refinement of stochastic models encompassing physical and/or seismological parameters (see e.g. Deodatis, 1996; Pousse et al. 2006, Spanos et al. 2007; Zerva, 2009; Rezaeian and Der Kiureghian 2010; Cacciola and Deodatis 2011; Cacciola and Zentner, 2012). It has to be emphasized, that those approaches currently proposed in the literature focus on the modelling of the free field ground motion, hence, without considering the influence of the urban environment. However, during an earthquake, a vibrating building emanates waves travelling through the ground over large distances. In an urban environment, the presence of several buildings in close proximity to one another generates the occurrence of multiple interactions that are referred to as seismic site-city interaction. Numerical studies on site-city interaction (see e.g. Clouteau and Aubry, 2001, Kham et al.,2006, Isbiliroglu et al. 2015), showed that the presence of buildings modifies significantly the energy of the seismic waves in the underlying soil layers resulting in decrement of the ground motion energy in some areas and increment in others. Therefore, the consequent ground-motion acceleration at the free-field used for designing civil engineering structures can be significantly different from the predicted one out-side the urban area or as so-called, free field motion. Several methods can be used to simulate site-city interaction effects. A recent review of structure-soil-structure interaction problem can be found in Menglin et al. (2011). Guéguen and Bard (2002) showed the effect of the city can be accounted for by modelling the structures as simple oscillators. Tsogka and Wirgin (2003) used omogenized blocks to study the seismic response in an idealized city. An homogenization method has been used also by Boutin and Roussillon (2004) to determine the multiple interactions between buildings. Groby et al. (2005) studied the seismic response of idealized 2D cities using a continuum viscoelastic medium. Ghergu and Ionescu studied the collective behavior of the buildings in a city like environment through a partial differential equation coupled with an ordinary differential equation through a special class of boundary conditions. More recently Isbiliroglu et al. (2015) used a finite element approach using parallel-computing code to simulate the ground motion during the 1994 Northridge earthquake and taking into account the coupled responses of multiple simplified building models located within the San Fernando Valley. The role of basing shapes and city density in the site-city interaction effect on the ground motion characteristics has been studied by Sahar et al. (2015). In this paper, a ground motion model for urban environment is proposed. The model aims to couple the traditional ground motion stochastic models defined at the free field and analytical attenuation law models to consider the impact of a vibrating structure on the surrounding free field ground motion. Verification of the proposed model is carried out by comparison of the results in terms of 50% fractile peak acceleration with pertinent Monte Carlo study conducted on a selected finite element model. Finally, an application shows the improvement of the prediction of the peak response acceleration with the propose ground motion model against the traditional approach based on the free field model.

2. Stochastic formulation of Ground Motion Model for Urban Environment

During an earthquake event, buildings of urban areas such as those shown in Figure 1, are experienced by dynamic vibrations induced by the joint effect of the free field motion of the soil deposit, $\mathrm{U_{FFM}}$, with the mutual interactions among foundations, $\mathrm{U_f(s,\omega)}$, where s is the spatial coordinate. The latter is referred to as Structure-Soil-Structure interaction (SSSI), or site-city interaction. Therefore, the seismic wavefield is altered by the presence of the buildings that can be interpreted as vibrating obstacles, inducing scattering to the ground motion waves. In order to analyse this phenomenon, the mechanical model illustrated in Figure 2 is herein proposed. The model is able to simulate the scattering of the seismic wavefield around a building by coupling a discrete model of a SDOF superstructure founded on compliant base with the analytical attenuation formulation proposed by Dobry and Gazetas (1988). The simple discrete model (Figure 2a) comprises a SDOF superstructure characterized by structural stiffness, k_{str}, and mass at the top of the superstructure, m_{str}, and a foundation-soil system, fully defined by stiffness, k_{SSI} for capturing soil-structure interaction effects and mass at foundation level, m_{f}. Only the horizontal absolute components of the structure and foundation displacements, U and $\mathrm{U_f}$, respectively, are considered by this proposed formulation.

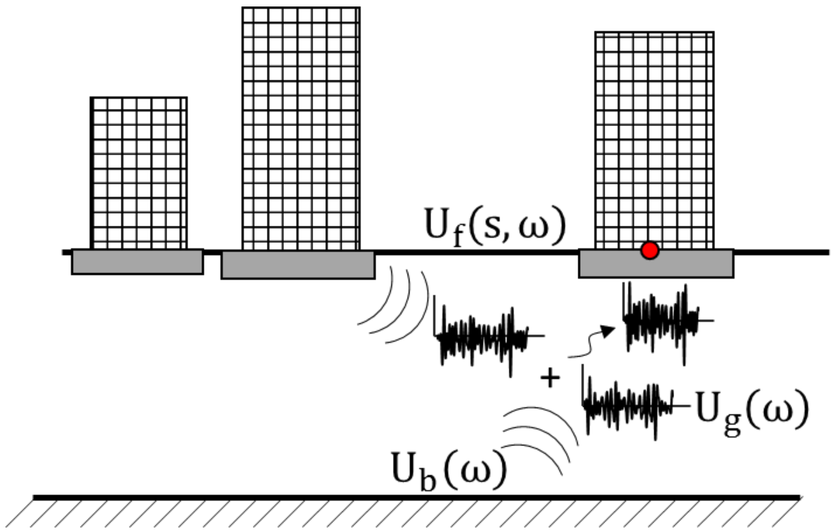

Figure 1 Seismic response of buildings in urban environment

Consider the discrete model of Figure 2a, the frequency transfer function of the foundation displacement, $\mathrm{U_f(\omega)}$, against the input motion at the base of the foundation, usually referred to as foundation input motion, $\mathrm{U_{FIM}(\omega)}$, is readily derived as follows:

$$H_f\ (\omega) = \frac{U_f(\omega)}{U_{FIM}(\omega)} = \frac{\tilde{\omega}_f^2(\omega^2-\tilde{\omega}_0^2)}{(\tilde{\omega}_0^2-\omega^2)(\omega^2-\tilde{\omega}_f^2)+\omega^2(\tilde{k}_{str}/m_f)} \tag{1}$$

where

$$\omega_0^2 = \left(\frac{k_{str}}{m_{str}}\right) \tag{2}$$

is the squared circular natural frequency of the fixed base SDOF superstructure and,

$$\omega_f^2 = \left(\frac{k_{SSI}}{m_f}\right) \tag{3}$$

is the squared circular natural frequency of the soil-foundation system. The use of the tilde accent indicates that the related quantity is amplified by the term $(1 + i\eta)$, where $i = \sqrt[2]{-1}$is the imaginary unit and, η is the loss factor. Once $H_f\ (\omega)$ is determined, the wavefield at ground level in the city, $U_g^c(S,\omega)$, induced by the motion of the foundation $U_f(\omega)$ at a certain distance S need to be determined. By approximating the foundation of the structure to an equivalent cylindrical shape the wavefield, $U_g^c(S,\omega)$ approximating the foundation can be expressed as follows

$$U_g^c(S,\omega) = \alpha(S,\omega)U_f(\omega) \tag{4}$$

in which $\alpha(S,\omega)$ is the attenuation function obtained by Dobry and Gazetas (1988) that adopt the formulation of the asymptotic cylindrical waves propagating from a cylinder subjected to harmonic signal defined by Morse and Ingard (1986):

$$\alpha(S,\omega) = \frac{U_g^c(S,\omega)}{U_f(\omega)} = \sqrt{\frac{r}{S}}\exp\left(-\frac{\eta\omega S}{V_w}\right)\exp\left[-i\omega\left(\frac{S}{V_w}\right)\right] \tag{5}$$

where t is the time, r is the equivalent radius of the foundation, η is the hysteretic damping of the soil, V_w is the velocity of the outgoing waves. The velocity of the waves, V_w, depends on the relative position of the considered field point as shown in Figure 2b; if the alignment between the foundation and the field point is perpendicular of the propagation of the travelling waves, i.e., it is in the upper or lower quadrant, $V_w \cong V_s$, in which V_s is the shear wave velocity of the soil, otherwise, if it is the left or right quadrant, $V_w \cong V_{La}$, where $V_{La} = (3.4V_s)/[\pi(1-\nu)]$ is the Lysmer's analogue velocity and ν is the Poisson coefficient.

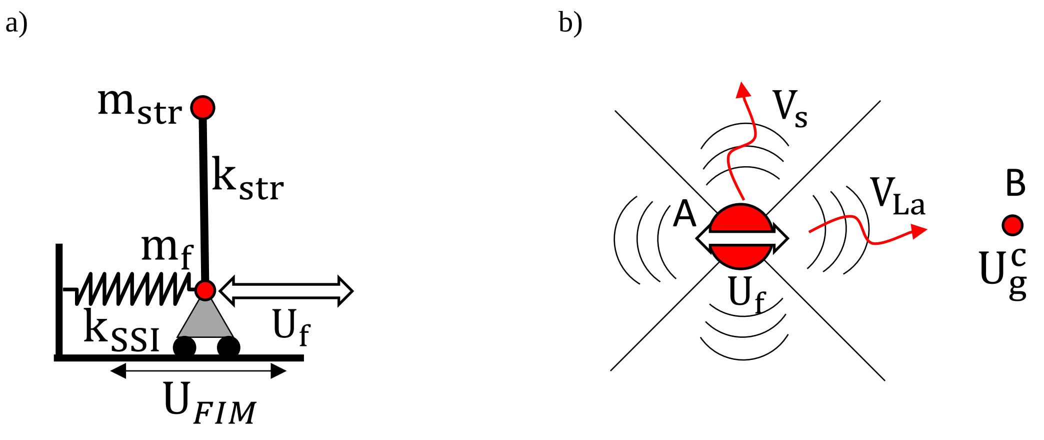

Figure 2 Coupled system to derived wavefield around a structure a) structural discrete model and b) attenuation model (after Dobry and Gazetas, 1988)

3. Stochastic response to Gaussian bedrock ground motion

Consider the scenario described in Figure 3 defined by structure at the location A subjected to the free field motion, $U_g(\omega)$. Because of the vibrations induced by the earthquake excitation, the structure emanates outgoing waves on the surrounding wavefield. At a certain distance S, the surface ground motion $U_g^c(S,\omega)$ is hence, given by the superposition of the free field motion at the point B, $U_g(\omega)$, and the scattered motion, $\alpha(S,\omega)U_f^{rel}(\omega)$ or equivalently, $\alpha(S,\omega)\left(U_f(\omega) - U_g(\omega)\right)$; therefore, the seismic wavefield $U_g^c(S,\omega)$ is expressed as follows:

$$U_g^c(S,\omega) = U_g(\omega) + \alpha(S,\omega)\left(U_f(\omega) - U_g(\omega)\right) \tag{6}$$

The absolute foundation displacement, $U_f(\omega)$, is derived by Eq. (1). It is worth emphasizing that for normalized frequencies, $a_0 = \frac{\omega r}{V_s} < 0.2{\sim}0.25$ or for shallow foundations (Jennings and J. Bielak, 1973,

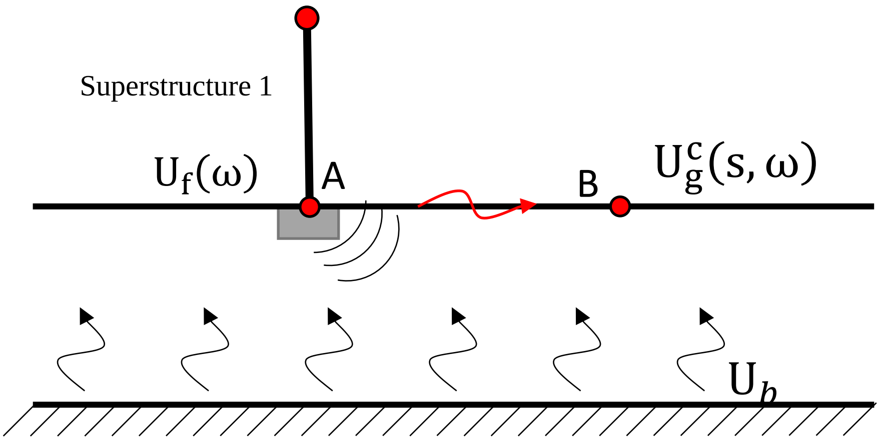

Figure 3 Seismic wavefield surrounding a building induced by bedrock excitation

Bielak, 1975, Wolf, 1985, Carbonari et al., 2018), the foundation input motion, $U_{FIM}(\omega)$, is assumed comparable with the free field motion, $U_g(\omega)$, i.e. $U_g(\omega) \cong U_{FIM}(\omega)$. Therefore, Eq. (6) can be rewritten by using Eq. (1) as follow:

$$U_g^c(S,\omega) = U_g(\omega) + \alpha(S,\omega)\left(H_f\ (\omega) - 1\right)U_g(\omega) \tag{7}$$

or equivalently:

$$U_g^c(S,\omega) = \left[1 + \alpha(S,\omega)H_f^{rel}(\omega)\right]U_g(\omega) \tag{8}$$

where $H_f^{rel}(\omega)$ is the transfer function of the foundation in relative displacement, defined as $H_f^{rel}(\omega) = H_f\ (\omega) - 1$.

The free field motion at the ground surface, $U_g(\omega)$, in Eq. (8), is obtained directly by adopting ground motion models for specific soil types or through local site response analysis in which the following relation holds:

$$U_g(\omega) = H_{Soil}(\omega)U_b(\omega) \tag{9}$$

where $H_{Soil}(\omega)$ is the transfer function of the soil deposit (Kramer, 1996) and $U_b(\omega)$ is the ground motion at the bedrock. The latter is herein modelled for simplicity sake by a Gaussian monocorrelated zero-mean stationary process defined by the power spectral density function (PSD) of the ground displacement $G_{U_bU_b}(\omega)$ Therefore, using Eq. (8) and Eq. (9) the stochastic ground motion wavefield is derived as follows:

$$G_{U_g^cU_g^c}(S,\omega) = \left|1 + \alpha(S,\omega)H_f^{rel}(\omega)\right|^2 |H_{Soil}(\omega)|^2 G_{U_bU_b}(\omega) \tag{10}$$

or, equivalently,

$$G_{\ddot{U}_g^c\ddot{U}_g^c}(S,\omega) = \left|1 + \alpha(S,\omega)H_f^{rel}(\omega)\right|^2 G_{\ddot{U}_g\ddot{U}_g}(\omega) \tag{11}$$

where $G_{\ddot{U}_g\ddot{U}_g}(\omega)$ is the ground motion acceleration power spectral density at the free field that can be determined using traditional models proposed in literature (e.g. Clough and Penzien, 1975), or by a spectrum compatible power models (e.g., see Cacciola 2010, and Giaralis and Spanos, 2012) . The power spectral density function defined by Eq. (11) defines the ground motion in the proximity of a vibrating building through a Gaussian stationary model. The more the distance S increases (and $\alpha(S,\omega)$ decreases) the smaller will be the influence of the vibrating building to the earthquake induced ground motion so to converge to the traditional free field ground motion models.

4. Stochastic response to incoherent bedrock ground motion

The proposed stochastic ground motion model in the proximity of a vibrating building of Eq. (11) is herein extended to take into account an incoherent ground motion at different point of the free field. The excitation at the bedrock induced by an earthquake event with source far away from the soil deposit is due to seismic waves travelling through the rigid bedrock underneath a soil deposit. This phenomenon referred to as wave passage effect, results in a difference of arrival time at separate ground surface locations and hence, to a loss of coherency of the ground motion. Therefore, the ground motion at the bedrock, $U_b(s,\omega)$, as a function of the spatial coordinate, s, is more conveniently expressed as Gaussian stationary stochastic vector process (see e.g. Deodatis 1996). Let us consider the scenario in Figure 4, $U_b(s,\omega)$ can be expressed as a vector of the two location taken into account, as follows:

$$\mathbf{U}_b(\omega) = \begin{bmatrix} U_b(0,\omega) \\ U_b(S,\omega) \end{bmatrix} = \begin{bmatrix} U_b^A(\omega) \\ U_b^B(\omega) \end{bmatrix}. \tag{12}$$

Therefore, a system of equations can be derived by using Eq. (6) at the location A ($s = 0$) and B ($s = S$) as follows:

$$\begin{cases} U_g^{c,A}(0,\omega) = U_f(\omega) = H_f(\omega)U_g^A(\omega) \\ U_g^{c,B}(S,\omega) = U_g^B(\omega) + \alpha(S,\omega)\left(U_f(\omega) - U_g^A(\omega)\right) \end{cases} \tag{13}$$

or in matrix notation under the assumption the soil transfer functions underneath point A and B are the same $H_{Soil}^A(\omega) = H_{Soil}^B(\omega) = H_{Soil}(\omega)$:

$$\begin{bmatrix} U_g^{c,A}(S,\omega) \\ U_g^{c,B}(S,\omega) \end{bmatrix} = \begin{bmatrix} H_f(\omega) & 0 \\ \alpha(S,\omega)H_f^{rel}(\omega) & 1 \end{bmatrix} \begin{bmatrix} U_b^A(\omega) \\ U_b^B(\omega) \end{bmatrix} H_{Soil}(\omega) \tag{14}$$

and after simple algebra the following expression is achieved:

$$\mathbf{G}_{U_g^c}(S,\omega) = \begin{bmatrix} H_f\ (\omega) & 0 \\ \alpha(S,\omega)H_f^{rel}(\omega) & 1 \end{bmatrix} |H_{Soil}(\omega)|^2 \mathbf{G}_{U_bU_b}(\omega) \begin{bmatrix} H_f\ (\omega) & 0 \\ \alpha(S,\omega)H_f^{rel}(\omega) & 1 \end{bmatrix}^* \tag{15}$$

where

$$\mathbf{G}_{U_bU_b}(\omega) = \begin{bmatrix} G_{U_bU_b}^{AA}(\omega) & G_{U_bU_b}^{AB}(\omega) \\ G_{U_bU_b}^{BA}(\omega) & G_{U_bU_g}^{BB}(\omega) \end{bmatrix} \tag{16}$$

is the power spectral density matrix of the ground motion at the bedrock and

$$\mathbf{G}_{U_g^c}(S,\omega) = \begin{bmatrix} G_{U_g^c}^{AA}(S,\omega) & G_{U_g^c}^{AB}(S,\omega) \\ G_{U_g^c}^{BA}(S,\omega) & G_{U_g^c}^{BB}(S,\omega) \end{bmatrix} \tag{17}$$

is the power spectral density matrix of the wavefield around a considered building.

Equation (15) can be expressed in terms of a correction the free field ground motion vector in analogy of equation (11) in the following form.

$$\mathbf{G}_{\ddot{U}_g^c}(S,\omega) = \begin{bmatrix} H_f\ (\omega) & 0 \\ \alpha(S,\omega)H_f^{rel}(\omega) & 1 \end{bmatrix} \mathbf{G}_{\ddot{U}_g\ddot{U}_g}(\omega) \begin{bmatrix} H_f\ (\omega) & 0 \\ \alpha(S,\omega)H_f^{rel}(\omega) & 1 \end{bmatrix}^* \tag{18}$$

So to be used in conjunction with well know stochastic ground motion vector processes (see e.g. Deodatis 1996; Zerva 2009; Cacciola and Deodatis 2011)

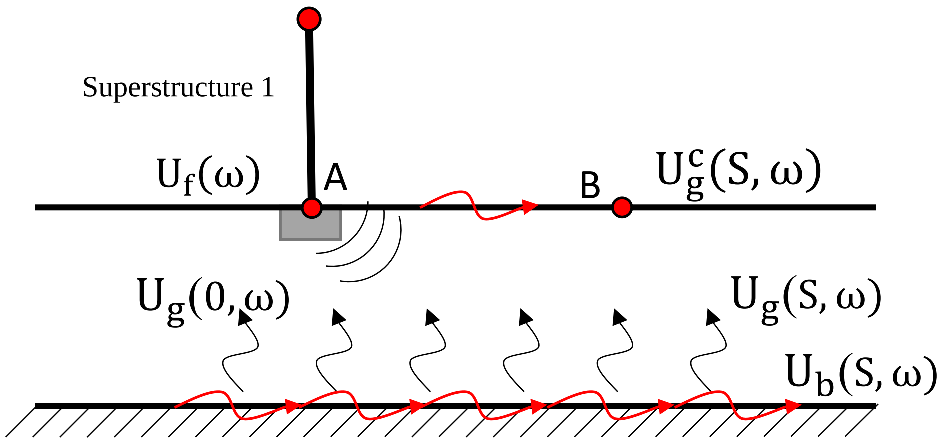

Figure 4 Seismic wavefield surrounding a building induced by incoherent bedrock excitation

5. Numerical results

5.1 Validation of the proposed analytical model

In this section the proposed ground motion model presented in Eq. (11) is validated through the study of the numerical finite element model illustrated in Figure 5. It comprises a superstructure located at the point A, defined by the stiffness, k_{str}, and mass, m_{str} founded on an embedded foundation of 1m-deep and 2m-wide, characterized by soil-foundation stiffness, k_{SSI}, and foundation mass, m_{f} . Values evaluated from the finite element model, are reported in

Table 1. The soil domain, characterized by average shear wave velocity, $V_s = 400\,m/s$, is 30m-deep and 800m-wide in order to avoid reflections of the waves on the lateral free boundaries of the domain. The soil domain is modelled with 9-Node Quadrilateral Elements under plane strain conditions without vertical degree of freedom. A Rayleigh-type damping is applied by considering a loss factor, $\eta = 0.1$, for both structural and soil domain. Seismic excitation is applied as prescribed acceleration to the bottom of the soil deposit. Monte Carlo simulation is performed by considering 15 Gaussian white noise signals with 500Hz cut-off frequency. The average PSDs obtained on the surface at the location A ($s = 0$) and B ($s = S$) are then compared to the proposed analytical PSD defined by Eq. (11) on Figure 6. It can be seen that the proposed curve matches well the average PSD obtained by MCS up to 10-15 Hz as mentioned in the previous section, i.e. for normalized frequencies $a_0 < 0.2{\sim}0.25$. This limitation is acceptable for seismic excitations since the energy is usually restricted to the range 0-10Hz. On the same Figure 6, the PSD of the free field motion $U_g(\omega)$ obtained from Eq. (9), indicated through a dashed line, is superimposed in order to visualize the effect of the wave scattering of the wavefield induced by the superstructure.

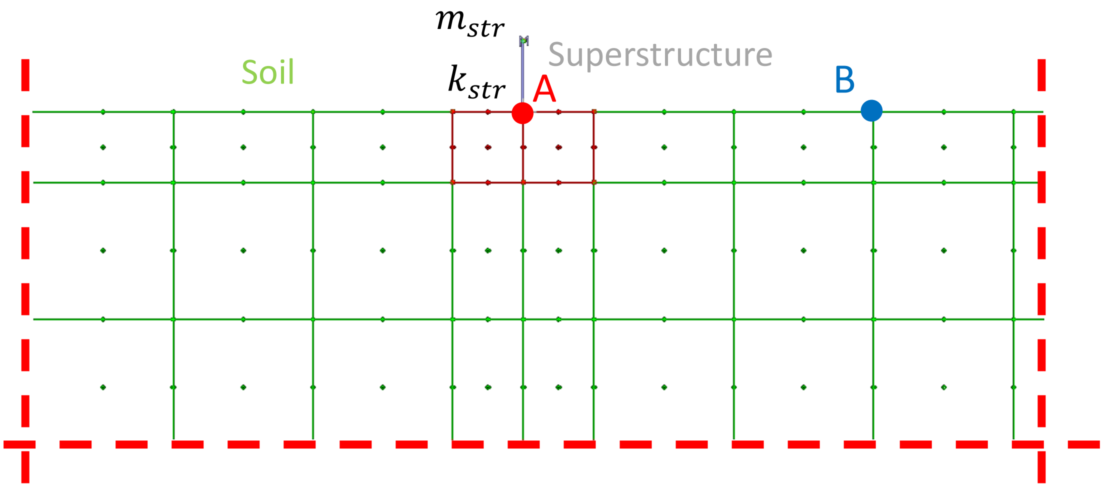

Figure 5 Close-up of the Finite Element Model used to analyse the impact of a structure on the surrounding soil

Table 1 Mechanical parameters used for the proposed discrete model

	k_{str}	m_{str}	k_{SSI}	m_f
Structure 1	$k_{str} = \omega_0{}^2 m_{str}$	Var.	693525300 N/m	15400 kg

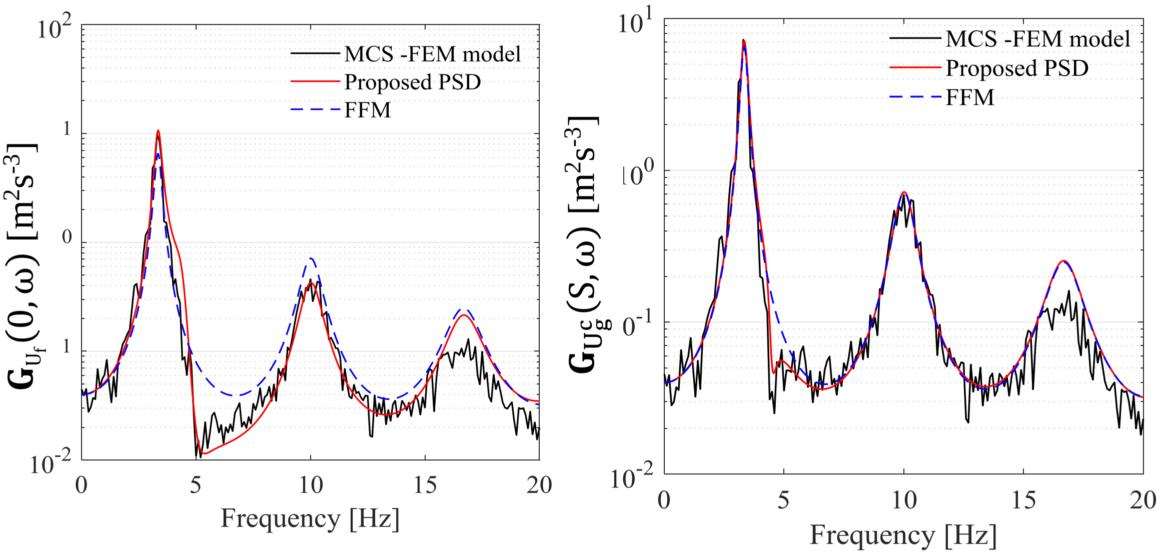

Figure 6 PSDs at a)point A and b) point B at 20m distance obtained by the proposed PSD and comparison with MCS

5.2 Parametric Analysis

In this section the influence of the structure on the surrounding soil is evaluated. The proposed ground motion model of Eq. (11) also depends on the structural characteristics of the superstructure through Eq. (1) such as the natural circular frequency ω_0 or equivalently, the structural period $\mathrm{T} = \frac{2\pi}{\omega_0}$ which can be made explicit in the proposed PSD such as $\mathrm{G}_{\mathrm{U_g^c U_g^c}}(\mathrm{T}, \mathrm{s}, \omega)$. In order to study the effect of the wave scattering as a function of the structural period T, the fractile of order *p* of the distribution response of peek acceleration of the wavefield is obtained by means of the first crossing problem:

$$\mathrm{X}_{\ddot{U}}(\mathrm{T},\mathrm{s}) = \eta_{\ddot{U}}\sqrt{\lambda_{0,\ddot{U}}} \tag{19}$$

where T_s is the time observing window; $\lambda_{0,U}$ is the zero-order response spectral moment and $\eta_{\ddot{U}}$ is the peak factor (see e.g. Vanmarcke and Gasparini 1977) given by

$$\eta_{\ddot{U}} = \sqrt{2\ln\left\{2N_{\ddot{U}}\left[1-\exp\left[-\delta_{\ddot{U}}^{1.2}\sqrt{\pi\ln(2N_{\ddot{U}})}\right]\right]\right\}} \tag{20}$$

with

$$N_{\ddot{U}} = \frac{T_S}{-2\pi\ln p}\sqrt{\frac{\lambda_{2,\ddot{U}}}{\lambda_{0,\ddot{U}}}} \tag{21}$$

and

$$\delta_U = \sqrt{1-\frac{\lambda_{1,\ddot{U}}^2}{\lambda_{0,\ddot{U}}\lambda_{2,\ddot{U}}}} \tag{22}$$

where the response spectral moments $\lambda_{i,U}$ are given by the following equation:

$$\lambda_{i,\ddot{U}}(\mathrm{T},\mathrm{s}) = \int_0^{+\infty}\omega^i\,\mathrm{G}_{\ddot{\mathrm{U}}_g^c\ddot{\mathrm{U}}_g^c}(\mathrm{T},\mathrm{s},\omega)\,d\omega. \tag{23}$$

An amplification index, AI, is then obtained by normalizing Eq. (19) to the response of the free field motion as follows:

$$\mathrm{AI}(\mathrm{s},\mathrm{T}) = \frac{X_{\ddot{U}}(\mathrm{T},\mathrm{s})}{X_{\ddot{U}_{\mathrm{FFM}}}(\mathrm{s})} \tag{24}$$

Figure 7 shows the normalized response, $\mathrm{AI}(\mathrm{s},\mathrm{T})$, of the 50% fractile of the peak acceleration (p = 0.5) for increasing distances, s from a vibrating structure with fundamental periods, T. In Figure 7a, the amplification index is obtained by considering a structure with mass, $m_{\mathrm{str}} = 200000\mathrm{kg}$; it worth mentioning that there are zones where the structure has a damping effect on the surrounding soil by decreasing the peak response and sectors where the structure induced a detrimental effect by increasing the maximum response of the soil. In particular, beneficial effects of about -11% are observed for natural structural periods close by the natural period of the site deposit ($\mathrm{T}_{\mathrm{soil}} = 0.3\mathrm{s}$) whereas for structural period shorter than the soil period amplifications of the response are obtained up to +30%, similarly as observed by Kobori et al. (1974). Figure 7b shows the same analysis done previously with a heavier mass, $m_{\mathrm{str}} = 500000\mathrm{kg}$; it could be observed higher deamplifications of the response for a broader range of structural periods and distances. Remarkable, reductions up to 23% are obtained for structural periods close to the natural period of the site deposit and amplifications higher than 90% of the free field motion are achieved.

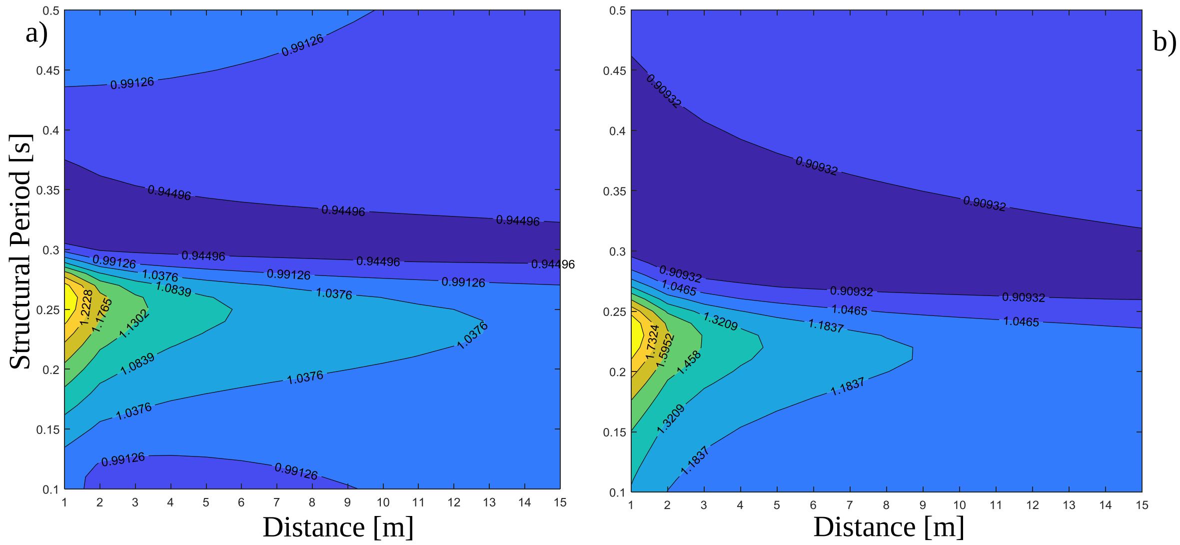

Figure 7 Normalized amplification wavefield of the 50% peak acceleration fractile around a structure with a) m_{str} =200,000kg and b) m_{str} =500,000kg

Use of the proposed ground motion model for structural design

Finally, an application of the proposed ground motion for structural design is shown. Consider the scenario in Figure 8 in which it is aimed to calculate the response of the structure 2 located at the point B influenced by the adjacent building 1 at location A placed at a distance, S = 3m. The two structures are characterized by the mechanical parameters given in

Table *1* determined for a structural period $\mathrm{T}^{A} = 0.2\mathrm{s}$ and mass $m_{\mathrm{str}}^{A} = 500000$ kg and structural period $\mathrm{T}^{B} = 0.5\mathrm{s}$ $m_{\mathrm{str}}^{B} = 200000$ kg for structure 1 and 2, respectively. A Monte Carlo Simulation with 30 stationary Gaussian white noises is performed. The average peak acceleration of the structure 2 at location B is equal to about 16.64 $\frac{m}{s^2}$. The 50% peak acceleration fractile obtained by the proposed PSD is 17.36 $\frac{m}{s^2}$, hence, with a small error of 4.33% with respect the numerical analysis because of the neglected interactions between the buildings.

It is worth noting that by using a conventional approach where the influence of the nearby vibrating building is neglected on the building, namely by considering the structure 2 by itself, the average peak structural acceleration would be 12.27 $\frac{m}{s^2}$, hence, 35.69% smaller than the actual dynamic peak response.

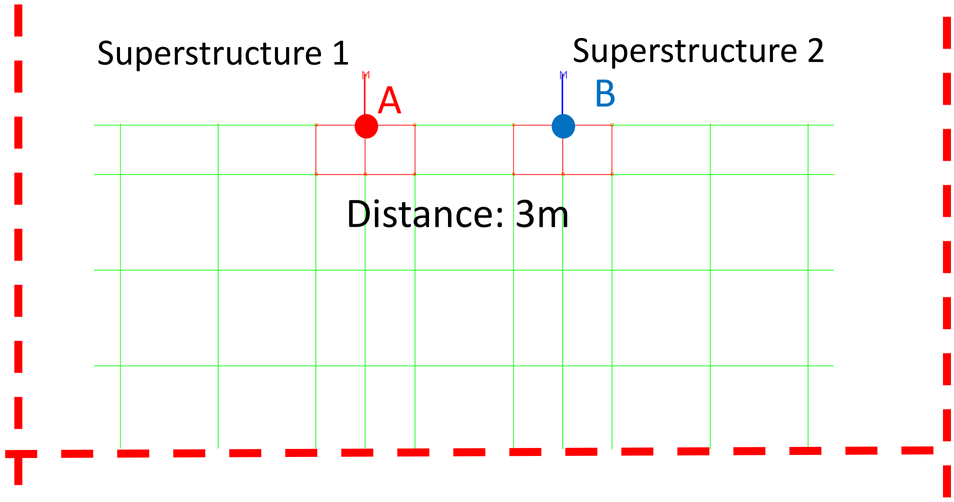

Figure 8 Close-up of the Finite Element Model used to analyse the dynamic response in urban environment.

Table 2 Analytical and Numerical results of the application on the proposed PSD for design in urban environemnt

Structure 2 (Location B)	MCS – coupled system	Analytical – coupled system	MCS – single structure	Relative Error using conventional procedure	Relative Error using proposed procedure
Median Peak acceleration $\frac{m}{s^2}$	16.64	17.36	12.27	-35.69%	-4.33%

6. Concluding remarks

In this paper, a stochastic ground motion model for simulating the seismic wavefield in the proximity of a vibrating building is proposed. A discrete model of structure on compliant foundation simulating the soil-structure interaction effects has been coupled with an analytical model of cylindrical wave propagation to capture the attenuation effects with distance of the vibrations induced by a building. It has been observed that during an earthquake, the joint effect of the seismic waves with these emitted scattered waves may produce amplifications or reductions of the free field motion. In particular, for small-medium distances, amplifications of the seismic wavefield has been observed when the structural natural frequency is lower than the natural frequency of the soil deposit, and vice-versa. Relevant amplifications over 90% of the free field motion have been obtained. Results obtained by the proposed ground motion model in urban environment has been verified through MCS of a numerical finite element model. Finally, an application of the proposed model has been carried out to show its relevance for seismic design purposes. The proposed model was able to well predict the dynamic response of a building collocated at a distance of 3m from another building with a small error of 4% while a relative error of about 36% has been obtained through traditional approach. This proves the importance of using specific ground motion models for urban environment for a reliable seismic design. Further studies will focus on the wavefield around more than two structures by extending the proposed model through a superposition approach.

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