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# Simple soil-structure interaction model for wind-induced vibrations in high-rise buildings

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**Keywords:** High-Rise Building, Wind-Induced Vibrations, Peak Acceleration, Soil-Structure Interaction, Stochastic Spectral Analysis

**Abstract.** The reliable design of high-rise buildings subject to wind-induced vibrations is important for ensuring comfort and safety. However, the Eurocode recommendations and common design practices used to estimate natural frequency and damping parameters in the design stage have been found to be inaccurate, leading to unreliable predictions of building accelerations. Previous studies have shown that soil-structure interaction (SSI) can have a significant influence on these parameters, especially for high-rise buildings on soft soils. This study presents a formulation of a simple model to include soil-structure interaction effects in the natural frequency and damping parameters. It also demonstrates the compatibility of this model with the current Eurocode procedures. The study compares the results of the simple SSI model with those of a continuous model of the New Orleans tower under design wind loading. In previous work, this continuous model was validated against in-situ measurements on the New Orleans tower. The results demonstrate that the simple SSI model significantly improves the accuracy of the predicted building accelerations under wind loading for the New Orleans tower, compared to accelerations determined in the original design and accelerations computed with the Eurocode guidelines.

## 1. Introduction

High-rise buildings (HRB) are sensitive to wind-induced vibrations. The occupant comfort under these vibrations is assessed based on the peak acceleration. This peak acceleration is strongly correlated with the natural frequency and the damping of the building. Bronkhorst and Geurts [1] showed that the damping and natural frequencies determined in the design phase of HRB can deviate significantly from measurements. Previous studies [2, 3] have investigated the influence of soil-structure interaction (SSI) on the damping of HRB, and observed that the overall damping is significantly influenced by soft soils. However, these studies did not establish the applicability of their models for the dynamic response under wind loading. Furthermore, they did not explore the influence of SSI on the peak acceleration under wind loading.

In-situ measurements of both wind loads and building accelerations performed on the residential tower New Orleans [1, 4] provide a good basis to investigate the influence of SSI on the wind induced dynamic behaviour of HRB. Carranza [5] developed a continuous model which consists of a frequency dependent description of the pile foundation, the building structure and the wind load. Carranza [5]



showed that this model provides an accurate estimate of the measured dynamic response of the New Orleans tower; the largest deviation between predicted and measured accelerations was 6%.

The results from [2], [3] and [5] show that the effect of SSI plays a significant role in the wind induced dynamic response (WDR) of high-rise buildings. Design procedures for the assessment of wind loads on HRB, such as EN 1991-1-4 [6], do not include the effects of SSI. This study investigates the effect of SSI on the wind-induced dynamic response, computed following the guidelines in EN 1991-1-4, and presents a simple procedure to include the influence of SSI.

This paper first provides a background of current design practices regarding the wind induced dynamic response of HRB in chapter 2. Chapter 3 presents the theoretical derivation of the procedure to include the influence of SSI in the Eurocode model. Chapter 4 presents the application of this model to the New Orleans tower. Chapter 5 presents the discussion of the results of the study. Chapter 6 gives conclusions and recommendations.

## 2. Background

### 2.1. Design practice

In design practice, the wind induced dynamic response of HRB is generally calculated using the procedures in the Eurocode EN 1991-1-4 [6]. This procedure requires the input of several parameters. This study looks into the natural frequency ( $f_n$ ) and damping ( $\zeta$ ) parameters.

For high-rise buildings the Eurocode gives the following approximations:  $f_n = 46/H$  and  $\zeta$  is dependent on the material of the structure (1.6% for concrete structures). However, in engineering design it is common practice to derive the natural frequencies with a FEM model. These models generally do not include SSI or include it in a simplified manner using elastic springs. The influence of energy dissipation in the ground and foundation is not included.

### 2.2. Comparison with measurements

Bronkhorst et al. [7] presented in on Dutch high-rise buildings that the current empirical damping estimators provide inaccurate damping predictions for high-rise buildings in the Netherlands. Studies by Smith and Willford [8], show that there is a tendency that the taller the building the lower the damping and the damping coefficient can get as low as 0.5%. This means that values such as the 1.6% proposed by the code for concrete structures can be significantly higher than the actual damping of the system and an overestimation in damping leads to an underestimation in acceleration. Moreover, both Bronkhorst et al. [7] and Smith and Willford [8], show results that suggest that for higher buildings, the structure material type plays less of an important role in the damping since measurement data show that varying the structure material type would not change the overall damping significantly. More analysis on these results and how they apply to this situation can be found in Carranza [5]. It is clear that the Eurocode approximation and other empirical approximations used in the design of buildings commonly overestimate the damping coefficient.

Carranza [5] used the measurements from Bronkhorst et al. [7] to compare the approximation for natural frequencies presented by the Eurocode and the ones used in the corresponding buildings design face. For this buildings the Eurocode approximation of the natural frequency had a wide spread of errors, from 35% underestimation to 17% overestimation with respect to measurements. This is to be expected since the simplified equation comes from the average result from empirical data but does not take into account that two buildings of the same height can have differences in footprint, materials, weight, stiffness ext. and therefore have significant different natural frequency's, consequently this approximation can prove unreliable.

However, even with the FEM models used in the design stage the natural frequency were 43% underestimated compared to the measurements. An underestimation of the natural frequency would lead to an overestimation of the wind loads. This results show that the FEM models predict a less stiff structure. This might be due to structural designers commonly overestimating the mass and underestimating the stiffness of the structure, since this most often leads conservative forces and deformations, for both gravitational and stability load cases.

### 3. Theory

#### 3.1. Wind-induced response model in Eurocode

Steenbergen et al. [9] explain the theory behind the model in EN 1991-1-4 to compute the wind-induced dynamic response. The structural model used in this study is a 2D system subjected to a 2D wind load, as depicted in Figure 1(a). The response of this system is described as a superposition of the vibration modes. Each mode can be described as a Single Degree Of Freedom (SDOF) system. This leads to the approximation of the building as a mass-spring-dashpot system on a fixed support, as depicted in Figure 1(a). The resonant response of the first mode commonly governs the maximum dynamic response of a HRB under wind load. This means that the dynamic response can be approximated by the response of the first mode subjected to a white noise spectrum representing the wind load. This study follows Procedure 2 in EN 1991-1-4, as formulated by Dyrbye and Hansen [10].

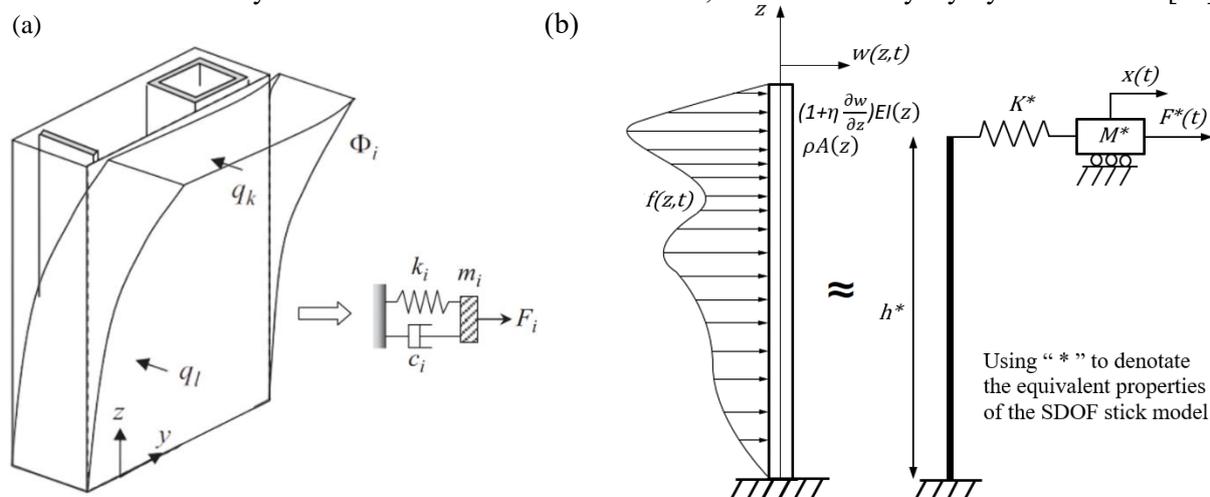


Figure 1. Depiction of equivalence between (a) 2D structure model and SDOF model. Image from Steenbergen [9], (b) Euler-Bernoulli model and SDOF stick model

Procedure 2 in EN 1991-1-4 [6] specifies for the standard deviation of the acceleration:

$$\sigma_{\ddot{w}} = c_f \cdot \rho \cdot I_v(z_s) \cdot v_m(z_s)^2 \cdot \frac{K_z \cdot K_y}{M/(H \cdot B)} \cdot \sqrt{K_s(f_n) \frac{S_L(z_s, f_n) \cdot \pi}{4 \cdot \zeta} \cdot \frac{\phi_1(z)}{\phi_1(z=H)}} \quad (1)$$

where the aerodynamic properties: \$c\_f\$, \$\rho\$, \$I\_v\$, \$v\_m\$ and \$z\_s\$ are respectively pressure coefficient, air density, turbulence intensity, mean wind velocity, reference height (\$z\_s = 0.6H\$). \$K\_s\$ is the size reduction factor for the wind load. \$S\_L\$ is the wind power spectral density function. The building properties: \$M\$, \$H\$ and \$B\$ are respectively the mass, height and width of the building. The \$\phi\_1\$, \$f\_n\$ and \$\zeta\$ are respectively mode shape, natural frequency and structural damping of the first mode. \$K\_y\$ and \$K\_z\$ are factors determine by the approximated mode shape (constant = 1, linear = 3/2, parabolic = 5/3).

#### 3.2. SDOF stick model

To include the effects of SSI the restoring forces and moments provided by the foundation are required. Section 3.1. introduces the Eurocode approximation of the building as a mass-spring-dashpot system on a fixed support. This model can describe the transverse forces on the fixed support but not the moments. To include the description of moments, this study uses a SDOF stick model shown in Figure 1(b).

The SDOF system is found by matching results of the first eigenmode of a fixed base Euler-Bernoulli beam model. Using this method it is possible to match up to four of the results of a continuous beam model to a SDOF system. Pruiksmá [11] presents a model matching: the natural

frequency, the top displacement, the base shear and the base moment. This is a good match for the SDOF model on this study since these four properties of the system are used in subsequent steps<sup>1</sup>.

The characteristics of the equivalent SDOF stick model, can be found with:

$$F^*(t) = L_1^h \cdot P_1(t) \cdot (M_1)^{-1} \quad (2) \quad L_1^h = \int_0^H \rho A(z) \phi_1(z) dz \quad (4)$$

$$h^* = L_1^\theta \cdot (L_1^h)^{-1} \quad (3) \quad L_1^\theta = \int_0^H z \cdot \rho A(z) \phi_1(z) dz \quad (5)$$

$$M^* = L_1^h \cdot [\phi_1(H)]^{-1} \quad (6) \quad M_1 = \int_0^H \rho A(z) [\phi_1(z)]^2 dz \quad (7)$$

$$K^* = \omega_n^2 M^* \quad (8) \quad P_1(t) = \int_0^H \phi_1(z) f(z, t) dz \quad (9)$$

with,  $\omega_n$  as beam's first natural frequency in rads/s.

The equation of motion of the equivalent system:

$$\ddot{x} + 2\zeta^* \dot{x} + \omega_n^2 x = \frac{F^*(t)}{M^*} \quad (10)$$

The wind load  $f(z, t)$ , is a stochastic load both in time and in space. This load can be simplified using the aerodynamic admittance  $\chi^2(\omega)$ , as a distributed force with a constant magnitude over the height, but varying magnitude in time:

$$f(z, t) \approx \mathcal{F}^{-1}[\chi(\omega)] \cdot q_w(t) \quad (11)$$

with,  $\mathcal{F}^{-1}$  as the inverse Fourier transform operator. Where  $q_w$  is the original wind load evaluated at reference height of the structure ( $q_w = f(z = z_s, t)$ ).

Combining equation (2), (9) and (11) the force  $F^*(t)$  of the equivalent system can be written as:

$$F^*(t) = \mathcal{F}^{-1}[\alpha(\omega)] \cdot q_w(t) \quad (12)$$

where,

$$\alpha(\omega) = \frac{L_1^h}{M_1} \chi(\omega) \quad (13)$$

The time fluctuating part of the wind load ( $q_w(t)$ ) which can be described as a stationary Gaussian process, is in the frequency domain defined by the spectrum  $S_{qq}(\omega)$ . The spectrum for the force  $F^*(t)$  can be written in in the frequency domain as:

$$S_{FF}(\omega) = [\alpha(\omega)]^2 \cdot S_{qq}(\omega) \quad (14)$$

The force spectrum  $S_{FF}(\omega)$  can be approximated with a white noise spectrum by evaluating equation (14) at the natural frequency.

The response of a SDOF system subjected to a stochastic white noise loading, can be separated into a background response and a resonant response. The building accelerations are mainly driven by the resonant response, as proved numerically by Steenbergen [12]. The standard deviation of the acceleration can be approximated with:

$$\sigma_{\ddot{x}} = \frac{1}{M^*} \sqrt{\frac{\omega_n \cdot S_{FF}(\omega_n)}{8 \cdot \zeta_n}} = \frac{\alpha(f)}{M^*} \sqrt{\frac{f_n \cdot S_{qq}(f_n) \cdot \pi}{4 \cdot \zeta_n}} \quad (15)$$

This equation computes the standard deviation of the acceleration at building height  $H$ . To calculate the acceleration at different heights, the mode shape of the first mode is used.

$$\sigma_{\ddot{w}}(z) = \frac{\sigma_{\ddot{w}}(z = H) \cdot \Phi(z)}{\Phi(z = H)} \quad (16)$$

For the case of a Gaussian narrow banded process, given the central frequency  $f_0$  and the period  $T$  (in this study  $T = 600$ s) of the event, the expected peak acceleration,  $\ddot{w}_e(z, T)$ , at a certain height  $z$  of a high-rise building can be calculated with:

$$\ddot{w}_e(z, T) = \sigma_{\ddot{w}}(z) \cdot \sqrt{2 \ln(T \cdot f_0)} \cdot (1 + 0.6 \cdot [2 \ln(T \cdot f_0)]^{-1}) \quad (17)$$

<sup>1</sup> Natural frequency is used for calculating the period lengthening, white noise spectrum and acceleration standard deviation. Top displacement is directly related to the maximum acceleration. Base shear and base moment are related to foundation stiffness through deformations.

### 3.3. SDOF stick model including SSI

The equivalent SDOF stick model is further modified to include the effects of SSI. The procedure is heavily inspired from the procedures described by NIST [13], which include the effects of SSI on the natural frequency and modal damping. The procedures, as introduced by NIST [13], were developed for seismic loading. This paper reformulates the influence of SSI on the SDOF stick model, to prove the approach is also valid for wind loading on high-rise buildings.

The SDOF stick model is linked to two dynamic impedance springs representing the foundation's translation and rocking stiffness and damping, denoted as  $D_t$  and  $D_r$ , respectively. Using “ $D$ ” to refer to these dynamic impedance springs, which consist of both damping and stiffness components. This model can be represented as the same mass attached to three springs in series. Through summation of reciprocals, it is equivalent to the mass supported by a single dynamic impedance spring ( $\tilde{D}$ ).

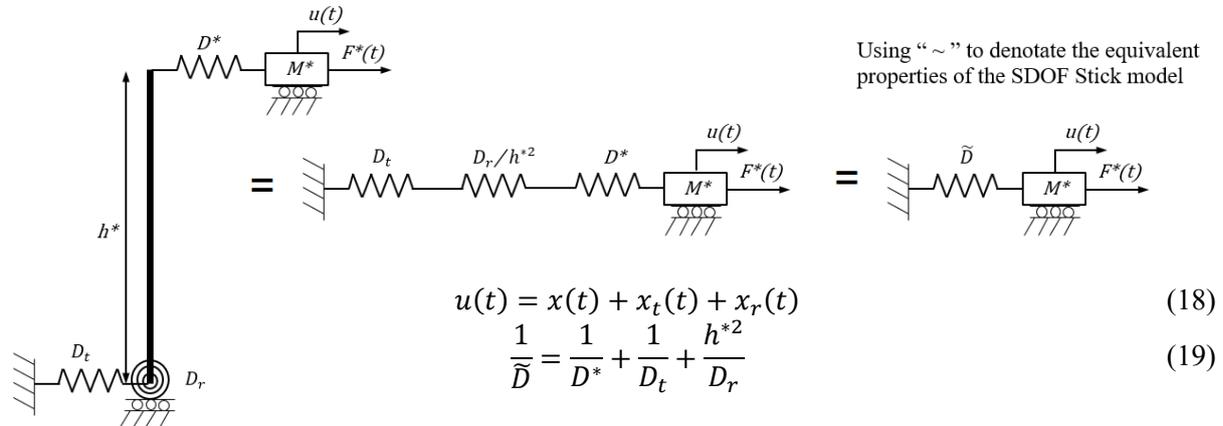


Figure 2. Depiction of equivalence between SDOF stick, springs in series and SDOF model.

To find the natural frequency of the system it is enough to analyse the undamped system. Where, the all dynamic impedance ( $D$ ) in equation (19) are simplified to the spring stiffness ( $K$ ) as,

$$\frac{1}{\tilde{K}} = \frac{1}{K^*} \left( 1 + \frac{K^*}{K_t} + \frac{K^* \cdot h^{*2}}{K_r} \right) \quad (20)$$

Calculating the natural frequency with  $\tilde{\omega}_n = \sqrt{\tilde{K}/M^*}$ , and comparing it to the original natural frequency of the SDOF stick model,  $\omega_n = \sqrt{K^*/M^*}$ , results in the same period lengthening equation as the one presented by NIST [13].

$$\frac{\omega_n}{\tilde{\omega}_n} = \frac{\tilde{T}}{T} = \sqrt{1 + \frac{K^*}{K_t} + \frac{K^* \cdot h^{*2}}{K_r}} \quad (21)$$

In a similar manner, the complex valued dynamic impedance of the equivalent system ( $\tilde{D} = \tilde{K}(1 + 2i\tilde{\zeta})$ ) can be calculated using the sum of reciprocals. Which, can then be separated into the real and imaginary components and formulated as stiffnesses and damping coefficients ( $\zeta$ ):

$$\frac{1}{\tilde{K}} = \frac{1 + 4\tilde{\zeta}^{*2}}{K^*(1 + 4\tilde{\zeta}^{*2})} + \frac{1 + 4\tilde{\zeta}_t^2}{K_t(1 + 4\tilde{\zeta}_t^2)} + \frac{(1 + 4\tilde{\zeta}_r^2)h^{*2}}{K_r(1 + 4\tilde{\zeta}_r^2)} \quad (22)$$

$$\tilde{\zeta} = \left[ \frac{\zeta^*}{K^*(1 + 4\tilde{\zeta}^{*2})} + \frac{\zeta_t}{K_t(1 + 4\tilde{\zeta}_t^2)} + \frac{\zeta_r \cdot h^{*2}}{K_r(1 + 4\tilde{\zeta}_r^2)} \right] \left[ \frac{1}{K^*(1 + 4\tilde{\zeta}^{*2})} + \frac{1}{K_t(1 + 4\tilde{\zeta}_t^2)} + \frac{h^{*2}}{K_r(1 + 4\tilde{\zeta}_r^2)} \right]^{-1} \quad (23)$$

Since,  $\zeta \ll 0 \therefore \zeta^2 \approx 0$ , equations (22) and (23) can be simplified to:

$$\frac{1}{\tilde{K}} = K^* \left( 1 + \frac{K^*}{K_t} + \frac{K^* \cdot h^{*2}}{K_r} \right) \quad (24)$$

$$\tilde{\zeta} = \zeta^* \left( 1 + \frac{K^*}{K_t} + \frac{K^* h^{*2}}{K_r} \right)^{-1} + \zeta_t \left( 1 + \frac{K_t}{K^*} + \frac{K_t h^{*2}}{K_r} \right)^{-1} + \zeta_r \left( 1 + \frac{K_r}{K^* h^{*2}} + \frac{K_r}{K_t h^{*2}} \right)^{-1} \quad (25)$$

These are the same formulas presented by NIST [13]. This derivation shows that these formulas are based on a system of viscoelastic springs in series and do not have any inherit assumptions that are only valid for seismic loading. In other words, these formulas can be used for high-rise buildings under wind loading.

### 3.4. Theoretical equivalence between SDOF models

The main difference between the starting points of the SDOF formulation on the Eurocode and the SDOF Stick formulation is that the Eurocode is a 2D wind and structure model while the Stick model is a 1D wind and Euler-Bernoulli beam model. Regarding the structure model, since this paper focuses on slender HRB they are governed by Euler-Bernoulli beam behaviour. Furthermore in terms of the Eurocode procedure, the 1D representation of the structure would be equivalent to a linear mode shape in the y-direction ( $K_y = 1$ ), which is commonly applied for HRB. Regarding the wind models, the 1D representation of the distributed wind load appears to lose the information of the y direction. However, in both scenarios the aerodynamic admittance is used to take into account the stochastic spatial correlation between the loads to an equivalent distributed load. The 2D wind load and aerodynamic admittance formulation is thus also compatible with the Stick model and can be used instead of the 1D formulation.

The other difference is in the geometry of the equivalent models. The Stick model mounts the exact same mass-spring-dashpot system to an infinitely stiff stick. The stick height being the lever arm between the transversal force and base, which is used to provide the base moment. Due to the stiff stick, all dynamic properties of the previous model stay the same, making the models equivalent.

Moreover, by rewriting the load  $S_{qq}(f_n)$  in its constituents (see Carranza [5]),

$$S_{qq}(f_n) = (B \cdot c_f \cdot \rho \cdot I_v(z_s) \cdot v_m(z_s)^2)^2 \frac{S_L(z_s, f_n)}{f_n} \quad (26)$$

the Stick model equation (16) can be written in a similar format as the Eurocode equation (1),

$$\sigma_{\ddot{x}} = B \cdot c_f \cdot \rho \cdot I_v(z_s) \cdot v_m(z_s)^2 \frac{\alpha(f)}{M^*} \sqrt{\chi^2(f) \frac{S_L(z_s, f_n) \cdot \pi}{4 \cdot \zeta} \cdot \frac{\phi_1(z)}{\phi_1(z=H)}} \quad (27)$$

which provides, the following relation between the equations (27) and (1) respectively,

$$\frac{\phi_1(H)}{\int_0^H \rho A(z) [\phi_1(z)]^2 dz} \cdot \sqrt{\chi^2(f)} = \frac{K_z \cdot K_y}{M/H} \cdot \sqrt{K_s(f_n)} \quad (28)$$

This same relation between the Stick model and the Eurocode model was found by Steenbergen et al. [9] when comparing the 2D theoretical formulation to the Eurocode model. Furthermore, Steenbergen et al. [9] found, that compared to the numerical evaluation of the theoretical formulation, the values for the parameters provided by the Eurocode in Procedure 2 provide very similar results. See also Dyrbye and Hansen [10], where the parameters for typical mode shapes (linear, parabolic and sinusoidal) for  $K_z$ ,  $K_y$  and  $K_s$  are initially proposed and solved for.

From this can be concluded that when applying these approximations to the SDOF Stick formulation, an equivalent result as the one presented in the Eurocode is reached. In other words the Eurocode procedure is derived from a model equivalent to the SDOF Stick formulation presented in section 3.2. This implies that the methodology used in section 3.3. to include the effects of SSI to the SDOF Stick model is also valid for the Eurocode formulation.

### 3.5. Implementation SSI in Eurocode wind-induced response

To implement SSI in the wind-induced response model of the Eurocode, the natural frequency  $f_n$  and damping ratio  $\zeta$  are replaced in equation (1) by  $\tilde{f}_n$  and  $\tilde{\zeta}$ , which include the influence of SSI:

$$\sigma_{\ddot{x}} = \frac{H \cdot B \cdot C_D \cdot \rho_a \cdot \bar{v}(z_s)^2 \cdot I_v(z_s)}{\frac{M}{K_z}} \sqrt{K_s(\tilde{f}_n) \frac{S_L(z_s, \tilde{f}_n) \cdot \pi}{4 \cdot \tilde{\zeta}}} \quad (29)$$

This section provides a procedure of how these equivalent parameters are defined. Furthermore section 2.2. showed that the damping and natural frequency parameters from both the Eurocode recommendations and common design practices, differ significantly to measurements. Because of this the procedure presented also include recommendations for the estimation of the damping and natural frequency for the baseline condition of the fixed structure.

### 3.5.1. Foundation parameters.

To calculate the period lengthening effects the translation and rocking stiffness, and damping parameters for the equivalent foundation needs to be determined. There are several foundation models having various levels of complexity and accuracy, such as: Winkler, Wolfe's Cone, empirical rigid foundation, macro-element, Boundary Element Method (BEM), Finite Element Method (FEM) and other models. The various levels of complexity for modelling the foundation, give the engineer more flexibility to choose the modelling approach depending on type of foundation and the level of accuracy that is required. Nevertheless, what is important is to use an appropriate model that can provide the translation and rocking stiffness and damping parameters for the equivalent foundation. In this paper an example is given using dynamic stiffnesses obtained with Dynapile [16].

The stiffness and damping parameters due to translation and rocking can be extracted from the dynamics stiffness matrix obtained from the Dynapile model with,

$$\begin{aligned} K_t &= \text{Re}[D_{tt}(f_n)] & c_t &= \text{Im}[D_{tt}(f_n)]/\omega & \zeta_t &= c_t/2\sqrt{K_t \cdot M^*} \\ K_r &= \text{Re}[D_{rr}(f_n)] & c_r &= \text{Im}[D_{rr}(f_n)]/\omega & \zeta_r &= c_r/2\sqrt{K_r \cdot h^{*2} \cdot M^*} \end{aligned} \quad (30)$$

Where,  $D_{tt}$  and  $D_{rr}$  are components of the dynamic stiffness matrix, representing  $\tilde{K}_{tt}$  and  $\tilde{K}_{yy}$ , respectively, as explained in Carranza [5].

### 3.5.2. Natural frequency.

Fundamentally to obtain the equivalent natural frequency ( $\tilde{f}_n$ , including SSI) the period lengthening effect from equation (21) is applied to the natural frequency of the fixed base model. The use of fixed base 3D FEM models and/or beams models is the recommended approach. Nevertheless, the modelling procedures should be scrutinized, since from the comparison of natural frequencies in section 2.2. it show that the FEM models used in design often underestimate the natural frequency which can lead to higher wind loads.

### 3.5.3. Damping coefficient .

Section 2.2. introduced that the Eurocode commonly overestimate the damping coefficient, because of this a different way of calculating the superstructure structural damping is proposed. The superstructure structural damping ratio ( $\zeta_s$ ) is estimated with the empirical damping predictor by Jeary [14], which takes into account the reduction of damping with the increase of flexibility of structures. The superstructure structural damping is analogues the damping on the equivalent SDOF stick model on fixed condition ( $\zeta^* = \zeta_s$ ). Studies by Gomez [15] and Bronkhorst et al. [7] simplified the equation even further to,

$$\zeta_s = \frac{\tilde{\omega}_n}{2\pi} + 0.15 \quad (31)$$

The equivalent damping  $\tilde{\zeta}$  (including SSI) can be obtained by applying equation (25) using the superstructure  $\zeta_s$ , the foundation translation  $\zeta_t$  and the foundation rocking  $\zeta_r$  damping parameters.

## 4. Case Study New Orleans tower

This chapter presents a case study to investigate the influence of the proposed procedure for including SSI effects in the Eurocode on the wind-induced response. First the case study is introduced, as well as the continuous model [5] used to represent the building. Finally, showing the setup procedures and parameters for the models for design level wind loads.

#### 4.1. New Orleans tower

The New Orleans tower is a residential high-rise building, located in Rotterdam since 2010. It has an architectural height of 158 m, and a structural height of  $H = 154$  m. Figure 3. Shows a top and side view drawing of the 44-storey high-rise (A) and 3-storey low-rise (B) part of the building. It is a concrete building cemented on concrete piles, with a two-storey basement. For this case study, the focus is only on the tower (A).

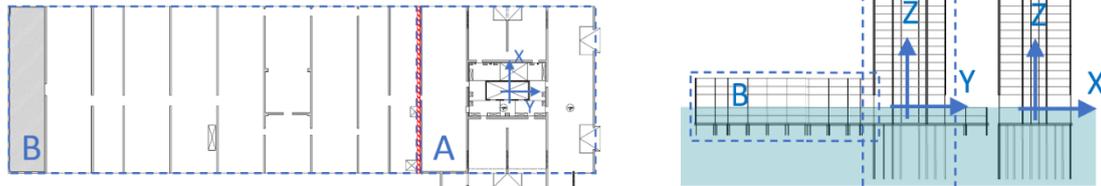


Figure 3 Elevation and plan view of the New Orleans tower, showing the sections and reference frame.

The New Orleans tower is instrumented with equipment to continuously measure the wind pressures, the wind velocity, and the building accelerations. The wind pressure and the building accelerations are measured at the 34<sup>th</sup> storey at a height of 114 m and the wind velocities on a mast on the roof of the building at a height of 156 m. Detailed information about the monitoring setup and the data acquisition can be found in [17, 18]. The in-situ measurements of both wind loads and building accelerations that are being performed on the residential tower New Orleans, provide a good basis to investigate the influence of SSI on the wind induced dynamic behaviour of high-rise buildings.

#### 4.2. Continuous model

This model consists of a frequency dependent description of the pile foundation, the building and the wind load; it requires the formulation of three different sub-models. The foundation sub-model, which provides the influence of SSI through a foundation dynamic stiffness matrix. The wind load sub-model, which provides the stochastic approximation for the wind loading. Finally, the tower structure sub-model, which incorporates the results of the foundation dynamic stiffness and the wind loading into its continues beam model to compute the dynamic response of the system. A depiction of these models is presented in Figure 4. The dynamic response of the system is computed in the frequency domain. Due to the loading being of stochastic nature, the dynamic response of the system is of stochastic nature as well. The sub-models, combination of these sub-models and the procedure to obtain the dynamic response and other relevant properties of the system are presented in Carranza [5].

##### 4.2.1. Model Setup.

The setup of the foundation model is based on CPT measurements, pile material properties and the pile distribution. The properties from the wind load come from wind pressure measurements grouped into events with similar magnitudes. The properties from the tower structure were based on the results of a 3D FEM model in SCIA Engineer. Parameters are presented in Carranza [5].

##### 4.2.2. Result comparison.

Carranza [5] used the acceleration measurements of the New Orleans to validate the continuous model of the tower. For the different wind groups, the highest deviation of the predicted accelerations compared to measurements was of 6%<sup>2</sup>. A comparison with the same model but assuming a fixed foundation (no SSI) show up to 27% underestimations of the acceleration. Results of Carranza [5] study are summarized in Figure 5.

<sup>2</sup> Results deviate slightly from the ones presented by Carranza [5] because it was found that: limits of integration for the measurements included the noisy low frequencies; and there was a typo on the EI of section 1 ( $32.6 \rightarrow 36.2$  [ $\times 10^{12}$  Nm<sup>2</sup>]). The change in natural frequency was only 2%.

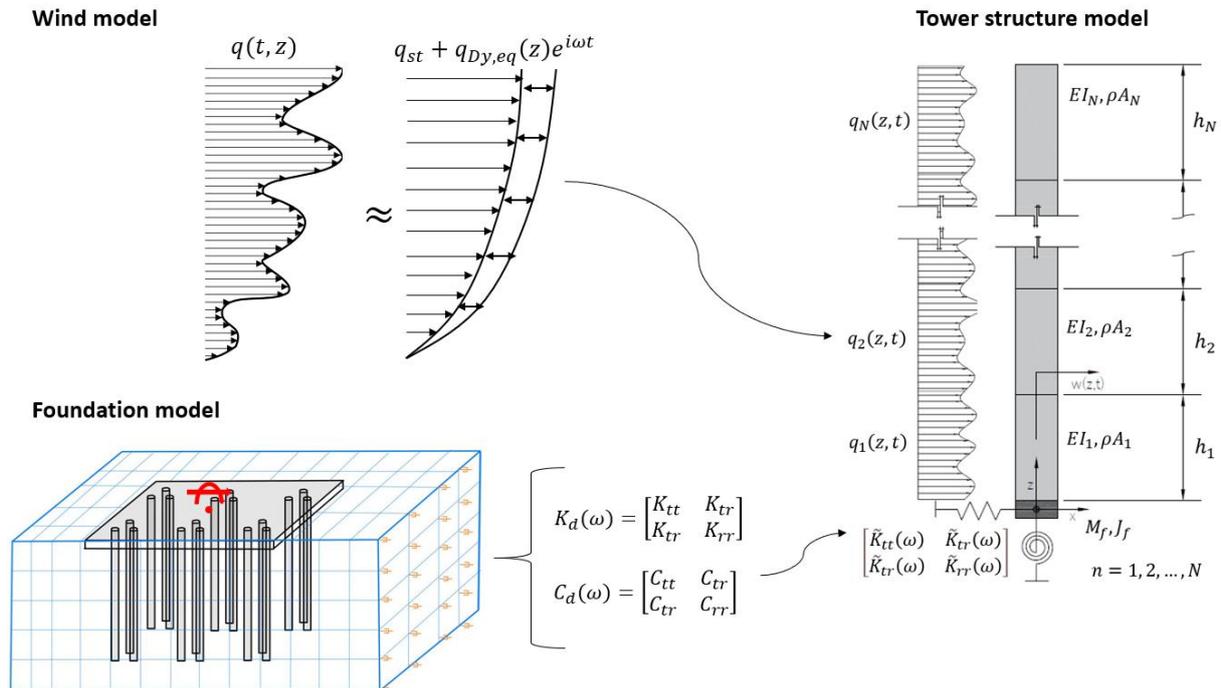


Figure 4. Depiction of the three-part (foundation, wind and tower) assembly of the continuous model.

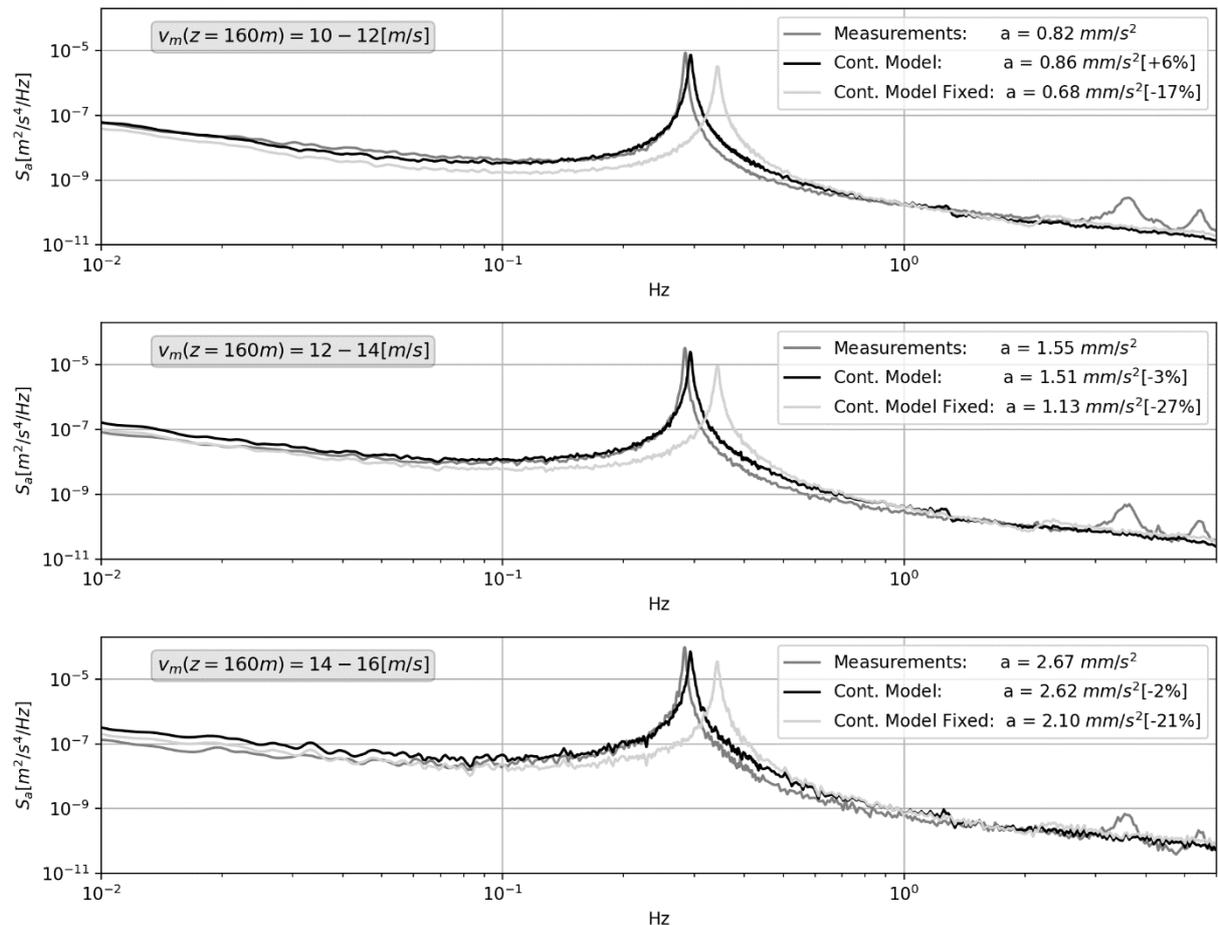


Figure 5. Acceleration response density function at 114m of the continuous model and measurements

#### 4.3. Set up of models comparison for design level wind loads

Since the Eurocode procedure relies on design wind load and cannot accommodate wind force measurements from the case study without potentially compromising the results, the continuous model for the New Orleans tower will be used as a comparison point. This model offers the necessary accuracy and flexibility in loading input to effectively serve as a reference for comparison.

##### 4.3.1. Set up for continuous model.

The continuous model maintains the same set up as for the comparison with the measurements, with the exception of using design wind loads. For the equivalent wind load,  $S_{q_{eqe}}(f)$ , from measurements it was necessary to use the wind load pressures at the measurement height to find the wind pressure at the reference height by using the theoretical distribution in height of wind (Carranza [5]). In the case of using the wind loading from the Eurocode the equivalent wind load can be calculated directly with:

$$S_{q_{eqe}}(f) = B \cdot C_D \cdot \rho_a \cdot \bar{v}(z_s)^2 \cdot I_v(z_s) \frac{S_L(z_s, f_n)}{f_n} K_s(f_n) \quad (32)$$

For the wind load a base wind velocity ( $v_b$ ) of 20.5 m/s is used. This wind speed is in the magnitude range for Serviceability Limit State (SLS) base wind velocity from the Eurocode [6,19] for the case study's location and representative to several Dutch cities.

##### 4.3.2. Setup of Eurocode models.

Four Eurocode models were set up.

- **EC model:** the standard Eurocode procedure without consideration of SSI.
- **EC+ model:** the same as the EC model but using the natural frequency from structural model.
- **ECd model:** the same as the EC model but with the natural frequency and damping ratio as applied in the original design of the New Orleans tower.
- **EC-SSI model:** uses the full procedure presented in this paper to include the effects of SSI.

For the properties of the EC model, the system parameters of the continuous model are used, with some slight adjustments to fit the EC model. The height ( $H$ ) and mass ( $M$ ) of the tower is the same as for the continuous model. The width of the tower ( $W$ ) is averaged to a single width. The natural frequency ( $f_n$ ) is taken as  $46/H$ . The superstructure structural damping ratio ( $\zeta_s$ ) is taken as 1.6%. A parabolic mode shape is used ( $\phi_1(z) = z^2$ ,  $K_z = 5/3$ ,  $G_z = 5/18$ ).

The properties of the EC+ and ECd models are the same as the EC model with the exception that; for EC+ the natural frequency is taken from the clamped structure from the continuous model, and for the ECd the natural frequency and damping parameters are the ones used from the original design of the structure, provided in Bronkhorst et al. [7] study.

The properties of the EC-SSI model are the same as the EC+ model but including the period lengthening effect on the natural frequency and the damping ratio as introduced in section 3.5. Also this model estimates the starting point of the superstructure structural damping with the empirical damping predictor by Jeary [14].

#### 4.4. Results from models at design level wind loads

The predictions of the dynamic properties at the highest point of the building for the continuous model and the 3 variations of the Eurocode model are shown in Table 1. This table also shows the percent deviation ( $\Delta$ ) with respect to the results from the continuous model.

Table 1. Results of dynamic properties of all models

	$f_n$ [Hz]	$\Delta_f$	$\zeta$ [%]	$\Delta_\zeta$	$\ddot{w}_e$ [mm/s <sup>2</sup> ]	$\Delta_{\ddot{w}}$
<b>EC</b>	0.30	+ 3%	1.60	+74%	43.1	-30%
<b>EC+</b>	0.34	+17%	1.60	+74%	37.3	-39%
<b>ECd</b>	0.19	-35%	1.00	+8%	81.4	+32%
<b>EC+SSI</b>	0.31	+ 7%	0.97	+ 5%	54.2	-12%
<b>Continuous</b>	0.29	-	0.92	-	61.6	-

The standard EC model shows a 30% underestimation of the expected peak acceleration. It is evident that this is mainly due to the 74% overestimation of the damping ratio. The natural frequency approximation from the Eurocode provides a prediction with only 3% overestimation. This might seem very accurate but as presented in section 3.5.2. when compared to measurements of 5 Dutch high-rise buildings this approximation can significantly over- and underestimate the natural frequency. The EC+ model provides a worse prediction than the EC model with a 37% underestimation of the expected peak acceleration. The Ecd model underestimates the natural frequency by 35% and slightly overestimates damping by 8%, which leads to an overestimation of 32% on the peak acceleration. The EC+SSI model shows a 12% underestimation of the expected peak acceleration which is the most accurate between the 3 models. These results show a significant improvement over the predictions using standard Eurocode procedure.

## 5. Discussion

The results from the models at the design level wind loads, showed significant underestimation of the expected peak acceleration from the standard formulation in EN 1991-1-4. However, calculating the natural frequency from accurate structure models with the fixed base but not including the effects of SSI can lead to even worse results. Furthermore, using the natural frequency and damping from the original design presented a model with underestimated natural frequency and slightly overestimated damping, which led to an overestimation of the peak acceleration. The same underestimation of natural frequency and overestimation of damping is present on other Dutch high-rise buildings studied by Bronkhorst et al. [3]. As demonstrated by this results, this can provide acceptable conservative predictions of accelerations. Nevertheless, having two poor predictions that cancel out each other's effects is not desirable. Finally, the results from the full procedure presented in this paper to include the effects of SSI show a significant improvement over the predictions using standard Eurocode procedure. From these results it is apparent that considering the effect of SSI in the natural frequency and damping contributes significantly to the reduction of the underestimation of the dynamic response of the equivalent SDOF system. Moreover, this procedure does not require significant additional work from a design perspective, since for this types of projects it is already common to have 3D FEM models of the structure and the foundation for other stages of design.

## 6. Conclusion

This paper has presented the derivation of a simplified model to include SSI for the calculation of wind-induced vibration in high-rise buildings. It was demonstrated from first principles that this model applies to wind loading, and is easily incorporated in the Eurocode wind-induced response model. The presented results for the New Orleans tower demonstrate that this SSI model improves the accuracy of the predicted building accelerations. It is recommended that the developed procedure is applied to a broader range of buildings to determine its applicability and effectiveness.

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