Effect of Soil-Structure Interaction on the Dynamic Behavior of Masonry and RC buildings

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SUMMARY:
The interaction between soil and structure (SSI) is a typical issue of structural and geotechnical engineering, still open regarding its practical applications. Further investigation is, hence, required to develop simplified but reliable methods to account for such a phenomenon in routine structural analyses.
Soil-structure interaction may play an important role (typically ignored) in the assessment of the dynamic behaviour of the structures in terms of frequencies and vibration modes. Clearly, to account for SSI a proper characterization of the foundation soils is required. In the present work, after a description of the state of the art on SSI, the authors present the results of several parametric analyses carried out for different types of Reinforced Concrete (RC) and masonry buildings; the analyses are aimed at estimating the effect of SSI on different structural typologies by means of approaches widely used in the literature.

Keywords: Dynamic behaviour, Soil-Structure Interaction, masonry building, RC building.

1. INTRODUCTION

The definition of the structural model represents the most important and demanding moment when designing or assessing a structure, since in this phase the passage from a complex reality to a simplified schematization, which should assure a satisfying level of reliability, is made. Nowadays, the development of more and more advanced design software implementing non-linear constitutive laws for materials and resistant mechanisms and able to carry out dynamic analyses, offers the possibility to take into account complex effects in structural modelling, such as the interaction between the structure and the foundation soil.
Soil-structure interaction is certainly a typical subject but, nevertheless, still new in its applicative aspects. Further studies are necessary for converting the research highlights in practical tools to be used in routine structural analysis.
In the present work, after a description of the state of the art on soil-structure interaction phenomena, the effect of SSI is evaluated by means of numerical parametric analyses carried out for some Reinforced Concrete (RC) and masonry buildings representing different structural typologies.

2. STATE-OF-THE-ART ON SSI

2.1. Soil-structure interaction

When the foundation soil is not a "rock" (conventionally, a material with shear wave velocity V_s higher than 800 m/s), the structure cannot be modelled with a fixed base, as it is usually made in current practice, but it is necessary to take into account the interaction with the foundation soil (SSI).
Under seismic actions, the interaction generated between the structure and the foundation soil can be distinguished in two different mechanisms, generally known as "kinematic" and "inertial" interaction.

Due to the first mechanism, the motion of the foundation is different from the motion of the soil in free field conditions (the foundation and the structure are assumed to be without mass). The second mechanism is originated because the inertial forces of the structure (included the foundation) are transferred to the foundation as shear and bending moments, which in turn will be transferred again to the surrounding soil. As stated above, soil-structure interaction leads to an increment of: (1) the fundamental period of the structure compared to the fixed-base solution; (2) damping of the system due to the amount of energy dissipated through the foundation soil (radiation damping).
Since both effects produce an elongation of the fundamental vibration period of the coupled system, and hence a reduction of spectral accelerations and seismic actions to be imposed on the structure, neglecting the SSI is usually considered safe in the structural design.
However, recent studies have enhanced cases for which soil-structure interaction, if neglected, can lead to unsafe design (Mylonakis & Gazetas, 2000; Jeremic et al., 2004).
In general, the importance of soil-structure interaction phenomena can be estimated by means of some synthetic parameters (Ciampoli and Pinto, 1995), such as:
a) the relative stiffness between the structure and the foundation soil, $1/\sigma$:

$$\frac{1}{\sigma} = f_{fix} \frac{h}{V_s} \tag{1}$$

where f_{fix} is the frequency of the fixed-base structure, h is the height of the structure, V_s is the shear wave velocity in the foundation soil;
b) the ratio $\frac{h}{r}$, where r is the radius of the equivalent circular foundation to which can be assimilated a shallow foundation with any shape, provided that it is characterized by a shape coefficient lower than 4:1. The criterion of equivalence is imported in some seismic codes, such as the NEHRP (BSSC, 1997). It can be intuitively seen that the higher are the parameters l/σ and h/r, more relevant is the SSI effect.
c) a third parameter combining the first two ones, ϕ:

$$\phi = \frac{1}{\sigma} \cdot \left(\frac{h}{r}\right)^{0.25} \tag{2}$$

which can be used for the evaluation of the limit condition, over which the SSI effects are negligible for the dynamic behaviour of the structure and its design. Literature studies (Veletsos, 1977) indicate that for $\phi \leq 0.125$ the effect of SSI can be neglected.
In order to evaluate the effect induced by SSI on the fundamental vibration period of the structure, it is usually adopted the scheme of a single degree of freedom (SDOF) system, defined by a mass m placed at a height h and connected to the ground by means of concentrated translational and rotational springs (for sake of simplicity any torsional effect is neglected), i.e. complex functions known as dynamic "impedances". If the foundation structure is stiffer than the foundation soil, the springs can be considered uncoupled each other.
According to this approach, the period of the coupled system, T_{comb}, can be evaluated using the formula proposed by Veletsos and Meek (1974):

$$T_{comb} = T_{fix} \cdot \sqrt{1 + \frac{k}{k_u} + \frac{k \cdot h^2}{k_\theta}} \tag{3}$$

where T_{fix} represents the period of the fixed-base structure, k is the stiffness of the fixed-base structure modelled as SDOF, k_u and k_θ are translational and rotational spring stiffness, respectively.
In general, k_u and k_θ depend on the frequency ω of the input and can be expressed multiplying the static stiffness, K_u or K_θ, by a dynamic coefficient , α_u or α_θ, (Gazetas, 1991):

$$k_u = \alpha_u(\omega) \cdot K_u \tag{4a}$$

$$k_\theta = \alpha_\theta(\omega) \cdot K_\theta \tag{4b}$$

For the estimation of parameters in Eq. 4, Gazetas (1991) provides different analytical expressions and/or charts, depending on the shear stiffness, G, and the Poisson coefficient, v, of the soil, the geometrical characteristics of the foundation and the frequency, ω, of the input.
Afterwards, in Mylonakis et al. (2006) a state-of-the-art is presented about different formulas proposed in the literature for the evaluation of SSI taking into account: 1) the shape of the foundation (circular, strip, rectangular/square foundation); 2) the soil model (homogeneous half-space, soil layer over rock); 3) the foundation embedment (foundation on the limit surface of the half-space or embedded in it).
The solutions found in literature are always obtained under the hypothesis of linear elastic behaviour of the foundation soil. Anyway, in presence of strong seismic actions, the soil does not show a linear elastic behaviour anymore and hence the elastic parameters of the soil should be accurately set. For example, some codes suggest estimating the SSI effect by means of strip-based methods, upper-bound and lower-bound types, assigning to the soil, in the first case, the maximum stiffness G_0, and, in the second case, the stiffness G_{eq}, compatible with the level of deformation that can be induced by the earthquake. Similarly, in presence of seismic actions at the ultimate limit state, also the structure will be in the post-elastic field and the ratio between the stiffness of the structure and the soil will be certainly modified.

2.2. Approach with concentrated springs at the base

In the estimation of the stiffness present in the Eq. 4 (in this stage, the contribution of the dynamic part has been neglected), different soil-foundation models can be assumed. In particular, the following schemes have been considered:
1) rectangular foundation resting on the limit surface of an elastic halfspace;
2) rectangular foundation embedded in an elastic halfspace;
3) equivalent circular foundation resting on a deformable soil layer (with finite thickness H) on rock.
For the sake of brevity, the analytical expressions of the static stiffness K_u and K_θ are omitted, referring to specific works (Gazetas; 1991).
In the following, Eq. 3 is reported showing the contribution of the two foundation impedances related to translational and rotational modes and the stiffness of the fixed base structure, K_{fix}:

$$T_{comb} = T_{fix} \cdot \sqrt{1 + \frac{K_{fix}}{K_{fond,trasl}} + \frac{K_{fix} \cdot h^2_{strutt}}{K_{fond,rot}}} \tag{5}$$

In Fig. 1 the ratio T_{comb} / T_{fix} is plotted as function of the ratios of the stiffness of the fixed-base structure to the stiffness of the translational and rotational springs, $K_{fix} / K_{fond,trasl}$ and $K_{fix} \cdot h^2_{strut} / K_{fond,rot}$. Compared to the fixed base condition ($K_{fix} / K_{fond,trasl} = 0$ and $K_{fix} \cdot h^2 / K_{fond,rot} = 0$), the presence of the springs at the base of the structure produces an increment of the vibration period of the structure. The curves show that when $K_{fix} \cdot h^2 / K_{fond,rot} = 1$, which means the deformability due to the rotational spring is comparable to the deformability of the fixed-base structure, the period of the structure increases by 50% even without the other spring ($K_{fix} / K_{fond,trasl} = 0$). The same evidence is found when $K_{fix} / K_{fond,trasl} = 1$. Moreover, it is observed that when one of the springs gives a high contribution to the global deformability of the structure ($K_{fix} / K_{fond,trasl} > 5$ or $K_{fix} \cdot h^2 / K_{fond,rot} > 5$), the effect of the other spring on the period vanishes.

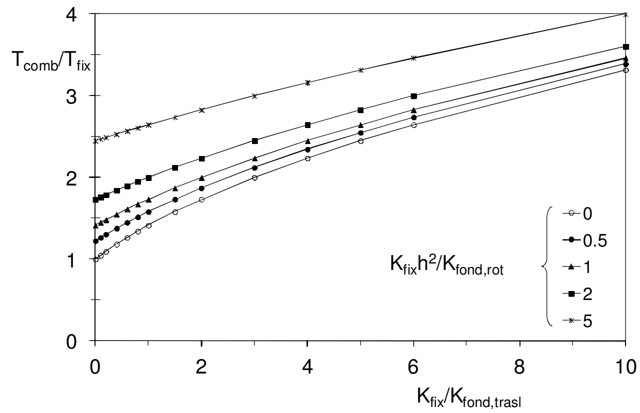

Figure 1. Effect of springs on period of vibration.

2.3. Approach with distributed vertical springs at the base (traditional Winkler approach)

In order to take into account soil deformability a widespread approach for practical structural design is based on the introduction of vertical springs uniformly distributed under the foundation structure. Such springs represent an elastic constraint to rotation of the structure and can reduce its overall stiffness and amplify its period of vibration. The deformability of the base constraint depends on shape and dimensions of foundation, but also on soil properties through the coefficient of soil reaction (ratio between load and vertical displacement), k [F/L^3]. The assessment of such a coefficient is an important and questionable aspect of the Winkler schematization. In the following sections, some examples of application of such a traditional approach are reported and are compared to the one with concentrated springs, discussed in section 2.2.

3. STUDY OF SSI EFFECT FOR SOME CASE STUDIES

3.1. Masonry palace

3.1.1. Description of the structure

The first case study is a masonry palace (named Palazzo Bosco-Lucarelli) located in Benevento (Italy). This building has a rectangular holed floor plan consisting of an underground floor, not completely accessible, a ground floor, two upper levels, and an attic under the pitched roof. The largest dimensions in plan are 33.5 m and 26.3 m and the total height is 18.2 m. The walls are made of different types of masonry (natural stones at the underground and ground floors, clay bricks at the upper floors). Thickness of masonry walls ranges between 0.6 m and 1.0 m with the greatest values at the ground floor. Floors are made by steel profiles (double 'T' shaped section with height of 0.2 m) spaced of 0.85 m and interspersed with brick tiles and covered by a concrete deck 0.1 m thick. At the ground floor, in the entrance hall some rooms are covered by masonry vaults. The roof is made by a steel truss with profiled steel sheeting externally covered with brick tiles.

The foundation soil was characterized by means of two drilled holes, one of which is accompanied with SPT test, and by means of a down-hole test. All tests were made nearby the building and evidenced that the foundation soil is made by a sequence of layer of sandy silt and silty sand soil until about 15 meters from the ground level. At higher depth conglomerates are found. At a depth of about 60 meters the blue-gray clay is, finally, found. The shear wave velocity in the soil volume influenced by the presence of the building can be estimated around 600 m/s.

The structure was modelled using three-dimensional ‘brick’ Finite Elements (FE) by means of DIANA software in order to carry out verifications under gravity loads, modal analysis and non linear static analysis (Ceroni et al., 2010).

Table 1. Palace Bosco-Lucarelli: periods of vibration of the first two modes

Young modulus of masonry	T_y [s]	T_x [s]
E_t = 1500 MPa E_{1-2} = 1200 MPa	0.267	0.238
E_t = 1900 MPa E_{1-2} = 1500 MPa	0.237	0.212
E_t = 2250 MPa E_{1-2} = 1800 MPa	0.218	0.195

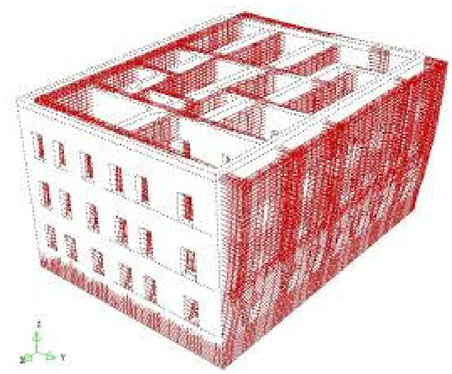

Figure 2. First vibration mode in Y direction for Palace Bosco-Lucarelli.

On the basis of in situ survey and the Italian code prescriptions (Min.LL.PP, 2008), for the different types of masonry present in the building, the following unit weights have been assumed: γ_t = 24 kN/m^3 for underground and ground floors and γ_{1-2} = 20 kN/m^3 for the first and second floor, while for Young modulus three couples of values have been considered, included within the ranges provided by the Italian code (Min.LL.PP, 2008). For modal analyses a coefficient of participation for variable loads of 0.6 in quasi-permanent combination has been assumed. In this condition the total weight is 88900 kN.
Linear modal analyses evidenced the first two modes of vibration, both characterized by participating mass ratio of 75% and by translational deformed shape: the first along Y direction (shorter side, see Figure 2) and the second along X direction (longer side). In Tab. 1 the periods of vibration of the structure for three couples of Young modulus, *E*, considered for the masonry of the ground and the upper floors are reported.

3.1.2. Soil-foundation-structure interaction with distributed Winkler springs

The first approach applied in order to take into account the soil deformability consisted in the introduction of vertical springs uniformly distributed under the walls of the ground level of the structure according to the traditional Winkler soil model. Linear modal analyses carried out with the FE three-dimensional model showed that, when the coefficient of soil reaction k_{wink} is 0.10 N/mm^3, the periods of vibration of the building matches the values found for the fixed-base scheme.
Moreover, the effect of the variation of k_{wink} and E on the global deformability of the structure has been studied by means of the refined FE 3D model. Regarding the Young modulus, it has been noticed that a reduction of about 33% gives an increment of the period of about 22%. Considering the variability of k_{wink}, it is observed that, when k_{wink} reduces of 33% (k_{wink} = 0.067 N/mm^3) with respect to the value assuring the fixed base response (k_{wink} = 0.10 N/mm^3), the period of the structure increases of about 20%. Therefore, the effects of the two parameters on the structural period are quite similar. This result is interesting if related to the reliability on the estimation of such parameters. In particular, the uncertainty in estimation of the masonry Young modulus is in general lower (less than 30%) than the one in evaluating the coefficient of soil reaction k_{wink}. For that reason, the employment of models with distributed springs according to the traditional Winkler model, if not well calibrated, can lead to wrong results with too high overestimation of the period of the structure.

3.1.3. Soil-foundation-structure interaction with concentrated springs at the base

In the following, the approach featured by the introduction of concentrated translational and rotational springs at the base is applied, in order to consider the soil-foundation-structure interaction and estimate the actual period of the structure (T_{comb}).
In Figures 3 the variability of the period of the structure with SSI, compared to the fixed base period, is reported in function of the previously introduced parameters V_s, l/σ and ϕ. The period of the structure with SSI is calculated according to the Eq.5. For the sub-system soil-foundation of Palace Bosco-Lucarelli three cases have been considered: shallow foundation on homogenous halfspace, embedded foundation in a homogenous halfspace, equivalent circular foundation in a layer over rock. This latter case better represents the actual conditions of the soil under the palace, where the first 10-

15 meters are more deformable compared to the deeper layers. For the foundations, an enlargement of the walls at the base of the underground floor with an embedment of 1.5 m has been assumed.
The variability range for soil shear wave velocity, V_s, considered in all the plots reported in Figures 3 is 300÷1500 m/s, where the lower limit corresponds to a reduced soil shear modulus G equal to ¼ of the initial value G_0 (correspondent to V_s = 600 m/s) in case of significant nonlinear effects, while the upper limit represents the case of an ideal rock.
In all models (for values of the Young modulus of the masonry equal to $E_{1\text{-}2}$=1200 MPa and E_t=1500 MPa), an increment of the period of the structure, T_{comb}, is clearly observed (see Fig.3a), for the combined effect of translational and rotational spring, when increasing the parameter l/σ previously introduced (Ciampoli e Pinto, 1995). These three curves give comparable values and it is observed that for V_s=600m/s, corresponding to l/σ=0.075, the increments of period, compared to the fixed base condition, vary between 12 and 18%, depending on the assumed model. In Fig. 4b the increment of the period of the structure considering the SSI effect is reported in function of the parameter ϕ. Considering an uncertainty of ± 120 m/s (±20%) on the mean value estimated for V_s within the significant volume of the building, the parameter ϕ ranges between 0.065 and 0.095, with an increment of the period of the structure between 5 and 25 % depending on the model assumed for the foundation. Therefore, the increments of the period can be not negligible also for values of ϕ below the limit suggested in the literature (ϕ =0.125 by Ciampoli and Pinto, 1995).

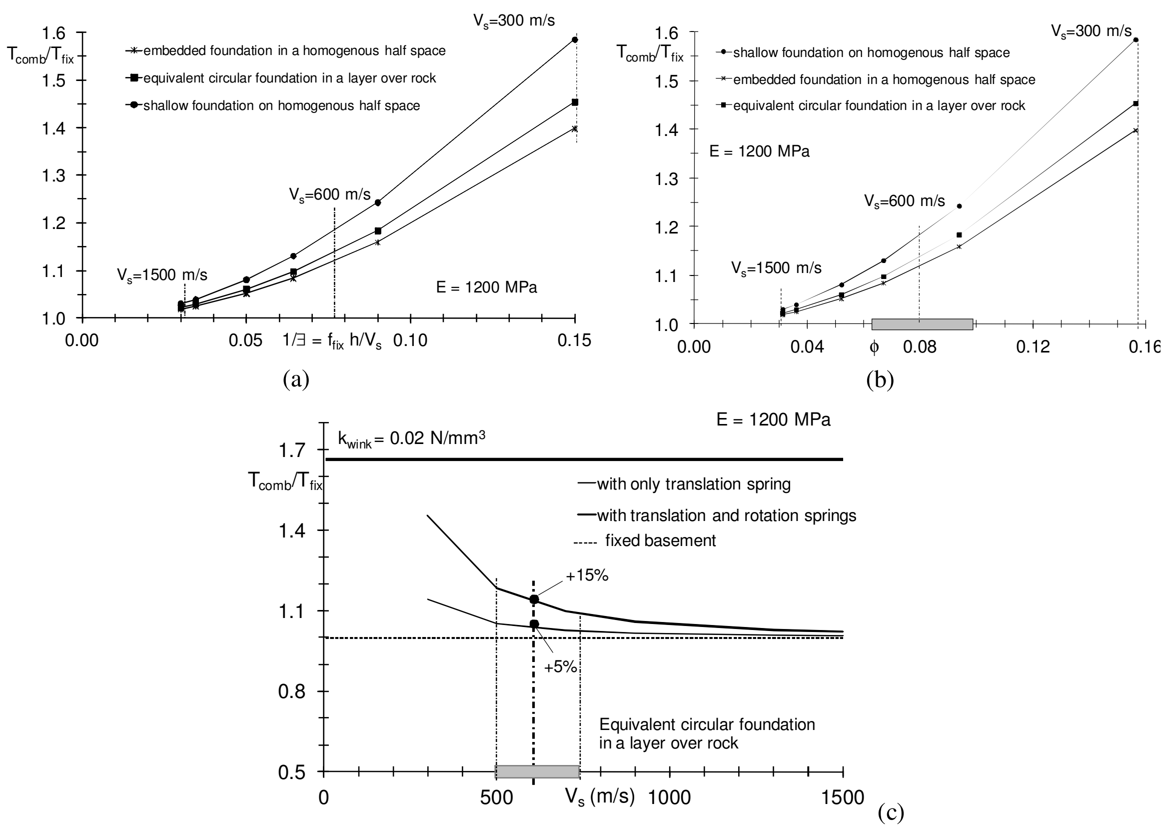

Figure 3. Variation of period of the structure with SSI in presence of concentrated springs:
a) vs. $1/\sigma$; b) vs. ϕ, c) vs. V_s.

In Fig. 3c the percent variability of the period of the structure with SSI compared to the fixed base is reported in function of V_s, differentiating the contribution to the global deformability of the translation spring from the combined effect of both translation and rotation springs. Note that in presence of the translation spring only, the Eq.5 has been used without taking into account the rotational term. In this analysis the model with equivalent circular foundation has been considered only, since it better represents the case of Palace Bosco-Lucarelli. Under this hypothesis and for the estimated velocity V_s = 600 m/s, the increment of the period is 5% due to the effect of translational spring only, and 15% in

presence of both springs. Considering an uncertainty of ±20% on the value of V_s, it has been observed an increment of period between 10 and 20% considering the effect of both concentrated base springs. The increment rises to 45%, if the lower limit of velocity V_s=300 m/s is considered.
In Fig. 3c it is also reported the increment of the period of the structure (about +65%) obtained considering the presence of distributed Winkler springs under each wall, when the coefficient of soil reaction k_{wink} is 0.02 N/mm^3 (equal to 20% of the value, 0.1 N/mm^3, assuring the fixed-base condition). The value of 0.02 N/mm^3 is in compliance with literature indications based on the foundation soil characteristics. The comparison shows that the increment in period obtained with the distributed Winkler springs is significantly larger (+65%) than the maximum increment obtained with the concentrated translation and rotation springs (+45%) in the unrealistic hypothesis of V_s=300 m/s.

3.2. Masonry Bell Tower

3.2.1. Description of the structure
The second structure considered in this study is the Bell Tower of the Carmine Church in Naples. The current structure from seventeenth century is founded on the structure of a former bell tower from XIV century, which remain is only the basement with a rectangular floor plan with maximum dimensions of about 8.0 m x 9.7 m (Fig. 4a). Up to 41 m the Bell Tower has a rectangular cross section with masonry walls made by yellow tuff. The upper part has octagonal cross section with walls made by clay bricks up to about 75 m. An octagonal room and a spire complete the structure up to a height of 68 m. The thickness of the walls is variable between 1 m and 4 m with decreasing thickness along the height. Even if the Bell Tower is located between other constructions, any structural connection can be excluded (Ceroni et al., 2009); the interaction with the adjacent church in X direction can be assumed due to contact constraints up to the height of 19 m, beyond which the Bell Tower is completely free.
On the basis of a detailed geometric survey, the structure has been modelled with 3-dimensional brick FE elements up to the height of 57 m, while the spire has been modelled with shell elements, by means of DIANA software in order to carry out the checks for gravity loading, linear modal analysis and non linear static analysis (Ceroni et al., 2010).
The unit weight of masonry has been assumed equal to 12 kN/m^3 for tuff in first 9 m, 11 kN/m^3 from 9 to 40 m, and 16 kN/m^3 for the bricks. Young modulus has been assumed equal to 900 MPa for tuff and 1200 MPa for clay brick on the basis of comparison with modal analysis results and dynamic identification by means of in situ tests (Ceroni et al., 2009). The total mass of the building is 33700 kN considering the permanent load only, since the structure is not open to the public.
Linear dynamic analyses showed that the first two vibration modes are the principal ones; in particular they are both characterized by a translational shape with a participating mass ratio of about 85%. The first modal shape is translational in X direction, parallel to the shorter side, with a period of 2.13 s and the second one is translational in Y direction, parallel to the longer side, with a period of 1.79 s.

3.2.2. Soil-foundation-structure interaction with concentrated springs at the base
For the foundation system at the base of the tower an enlargement of the walls at the base and an embedment of 1.5 m have been assumed. The soil model assumed for calculation of seismic impedances and period according to Eq. 5 is a shallow foundation on a homogeneous halfspace. In order to carry out parametric analysis to be compared to the ones previously performed for Palace Bosco-Lucarelli, the range of velocity considered is the same (V_s=300-1500 m/s), also because detailed information about the foundation soils are in this case not available.
In Fig. 4b, c, and d the variability of the period of the structure with SSI, compared to the fixed base period, is reported in function of the previously introduced parameters V_s, l/σ, and ϕ.
Fig. 4b shows that for the case of the Bell Tower the translation spring has a negligible effect on the structure period, even for very low shear wave velocity (V_s=300 m/s), due to the high deformability of structure itself. On the contrary, a very relevant effect is obtained accounting for the rotational spring, with an increment in period ranging from 5% to about 70%, depending on the shear wave velocity. Moreover, it is observed that for the Bell Tower the variability of the period with V_s is higher than for Palace Bosco-Lucarelli where, for the same lower limit (300 m/s), the maximum increment of period was 45% (see Fig. 3b). This different influence of the rotational spring for the Bell Tower is due to the high height of the structure, which has to be squared in Eq. 5.

In Fig. 4c the variability of Bell Tower's period with the parameter $1/\sigma$ is shown for both principal directions of the structure. It is observed that in Y direction (characterized by a period 15% lower than the one in X direction) the period of the structure is slightly more sensitive to variation of V_s.
Similarly, in Fig. 4d the variability of the period with the parameter ϕ shows that, in order to achieve a period increment of 20% in both directions, ϕ has to range from 0.10 and 0.12. These values are slightly higher than the one (ϕ = 0.095) estimated for Palace Bosco-Lucarelli to obtain the same increment of period.

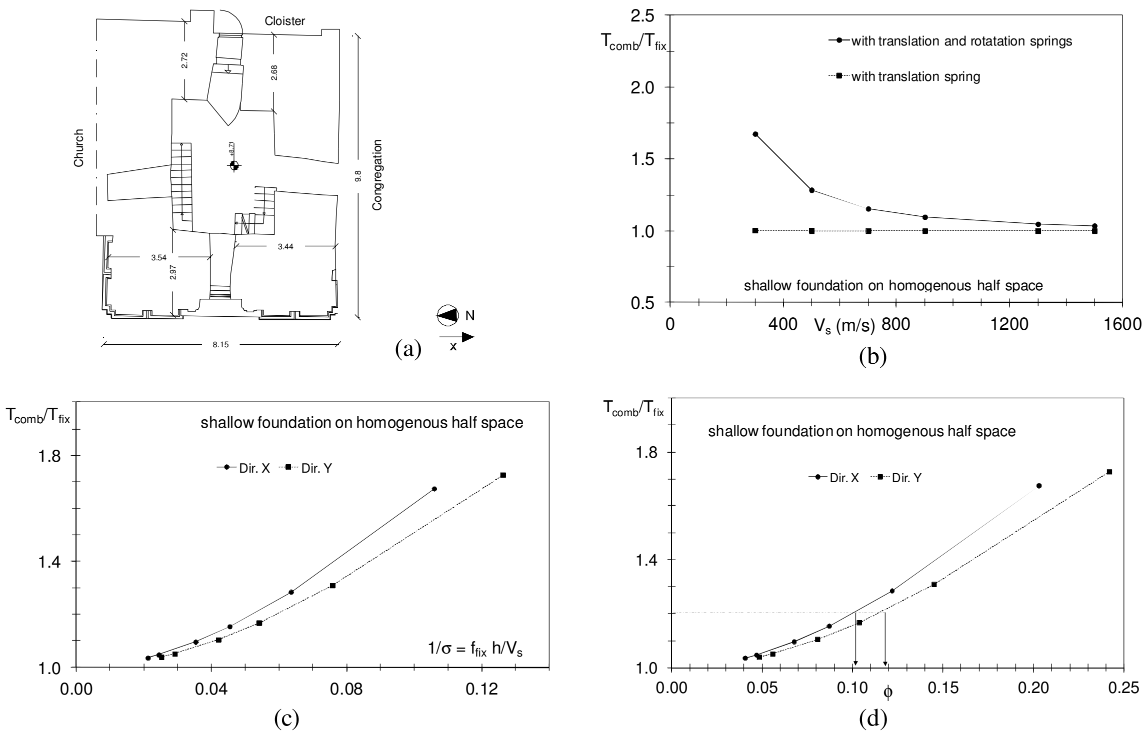

Figure 4. Bell tower of Carmine: a) ground floor plan; b) Variation of period vs. V_s; c) vs. $1/\sigma$; d) vs. ϕ.

3.3. Study of SSI effects on RC buildings

Further analyses for the evaluation of SSI effects have been carried out on different typologies of RC buildings. In particular, six buildings have been considered with different floor plan dimensions and structural typology (RC frames with beam and columns, RC frames with shear walls).
The first two buildings have dimensions of 20 m x 30 m and almost the entire participating mass is associated to the first two translational modes of vibration; in particular, the first building typology has a structure made by RC frames, while the second one is made of RC shear walls.
The second group of buildings has smaller dimensions in plan (22 m x 11 m) and features some irregularities in plan and/or elevation. For this group the participating mass ratios related to the first two translational modes range between 60 and 72%.
These first five buildings are designed for seismic areas according to Eurocode 8 (EC8, 2004) for an expected Peak Ground Acceleration PGA=0.2 g. The sixth building is an existing RC building with dimensions of 55 m x 27 m and height of about 21 m. The first mode of vibration is a translational mode along Y direction (shorter side) with a participating mass ratio of about 69%. The following modes are characterized by lower participating mass ratios in both directions, so that for the parametric analyses the first mode has been considered only. For all the buildings the foundation structure is made by a grid of RC beams with T shaped cross section. In Tab. 3 the main characteristics of the analyzed buildings are listed along with the principal periods of vibration in two directions and the corresponding participating masses. Names of axis are set with the same criterion for every building: the Y axis is parallel to the shorter side, while the X axis is parallel to the longer side. Referring to the period of vibration of these buildings in Y direction, in Fig. 5 the curves of

variability of the period in presence of SSI are reported considering the introduction of concentrated translation and rotational springs at the base, according to Eq. 5. The model of shallow foundation on homogeneous halfspace has been assumed, varying the value of V_s in the same interval considered in previous cases (300-1500 m/s).

Table 3. Properties of RC buildings

Type		T [s]		M [t]	
		Y	X	Y	X
20 m x 30 m H=12 m	1. Frame RC building	0.567	0.522	2085	2085
	2. Shear wall RC building	0.071	0.055	1850	1850
22 m x 11 m H=13.5 m	3. Regular frame RC building	0.55	0.52	491	491
	4. Irregular frame RC building in plan	0.53	0.51	371	371
	5. Irregular frame RC building in plan and elevation	0.50	0.46	308	269
55 m x 27 m H=21 m	6. Existing RC building	1.06	-	6983	-

It can be noticed that the interaction effect is negligible (lower than 5%) for buildings with periods between 0.5 and 0.6 seconds, regardless of their structural typology and for the whole considered range of variation of V_s, while it is more relevant for the shear walls building (no. 2) that is characterized by a lower period (0.07 s). Indeed, for the former cases the parameter ϕ is lower than 0.10 even for very low shear wave velocities of the soil; on the contrary, for the shear walls building (no. 2) ϕ varies between 0.11 and 0.55 depending on V_s. A period increases of 20% is achieved for ϕ= 0.18 that corresponds to V_s = 800 m/s. Also for the existing RC building (no. 6) the effect of SSI is negligible, even though it is taller than the other RC buildings (about 21 m vs. 12-13.5 m). In this case, the higher period of the building (T ≈ 1 s) and its higher deformability seem to reduce the rotational effect, which, mainly, depends on the height in Eq. 5.
In general, for RC buildings the period of vibration is considered to be strongly dependent on their height, as it is reported in formulations present in several standards and literature studies (Eurocode 8, 2004; NTC 2008; Hong and Hwang, 2000; Verderame et al., 2009). Parametric analyses on RC frame buildings (Verderame et al., 2009) by varying the number of floors, floor plan extension and presence of frames along one direction (building designed against vertical loads) or in two directions (buildings designed against seismic actions) showed that for the latter ones, which are characterized by a more uniform distribution of resisting systems in plan, the periods in the two directions are similar (maximum difference of 20%) and, thus, it can be assumed that the effect of SSI are comparable in two directions. Moreover, since the periods of frame buildings designed against seismic actions are lower than those of buildings designed against vertical loads only, the former could be more affected by SSI. Finally, being the periods of shear walls buildings lower than the ones of similar frame buildings, the effect of SSI can become important, as evidenced by the previously reported parametric analysis.

4. SUMMARY OF NUMERICAL ANALYSIS AND CONCLUSIONS

In Fig. 6 the variation of the structure period accounting for SSI is reported in the direction parallel to the shorter side for the RC frame building no. 1, the RC shear walls building no. 2, for the masonry building, and for the Bell Tower, varying V_s in the range 300-1500 m/s. It can be noticed that the increment of period can be considered relevant (higher that 10%) for V_s between 300 and 800 m/s for all the buildings, apart from the RC frame one (no. 1), in spite of the difference in the type of structure and geometry.
Moreover, it is observed that the influence of SSI is more relevant when the period reduces and when the stiffness of the structure increases (RC shear walls building is stiffer than the masonry walls building and much stiffer than the RC frame building) due to the combined effect of translation and rotational springs simulating soil deformability.
The effects of SSI can become, anyway, relevant also for very deformable buildings, characterized by

long periods if the building is also very tall, as in the case of the Bell Tower, since the height amplifies the contribution of the rotational spring in Eq. 5.

The values of parameter ϕ for which the effect of SSI are not negligible (increment of 20% in the period) are variable between about 0.1 (masonry building) and 0.2 (RC shear wall building).

Therefore, it can be concluded that the SSI effect cannot be neglected a priori, but it depends on some characteristics of the structure (stiffness, height) and on the foundation soil properties (V_s).

For that reason, by carrying out more detailed analyses on different types of buildings, it will be possible to better define the range of variability of global parameters for assessing relevance or not of SSI effects on structural analysis.

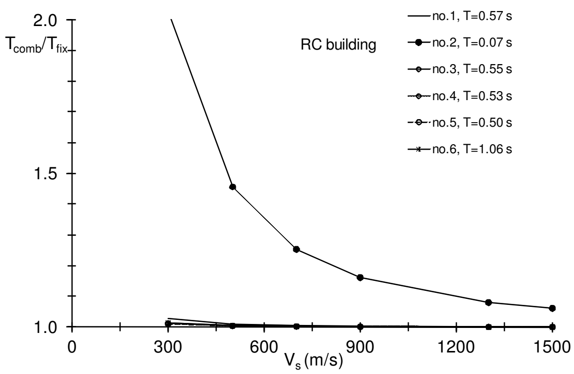

Figure 5. Variation of the period of vibration of RC buildings with SSI vs. V_s.

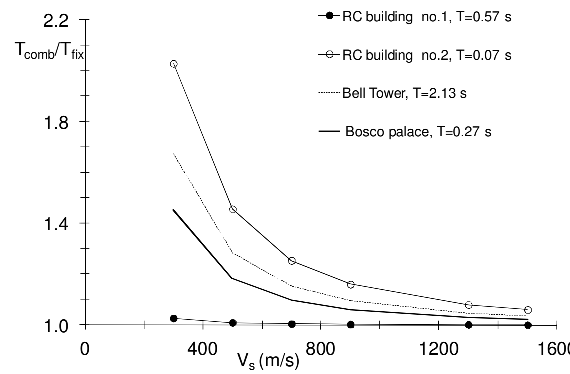

Figure 6. Variability of period of vibration of different typologies of buildings with SSI vs. V_s.

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