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Dynamic response of a rigid foundation rested in a densified soil

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ABSTRACT

A three-dimensional finite element numerical model is proposed to study the dynamic behavior of foundations placed on densified soils. The densification of soil was done by dynamic compacting. The characteristics of densified soil are organized in four soil classes according to Eurocode 8(NF EN 1998). The mechanical parameters of the densified soil are used to determine the dynamic response of the foundations before and after the densification process. The formulation is based on the sub-structure method, the characteristic of the soil and the foundations are considered viscoelastic. Dynamic excitations are applied to the center of the foundation; the displacement response is obtained by applying a vertical and a horizontal load to the centers of the foundations. The obtained results are presented in terms of vertical and horizontal displacement amplitude for all categories of densified soil.

1 Introduction

The soil compaction techniques are widely used to improve the mechanical characteristics of soil in a significant way [1–3]. The soil compaction permits to improve some parameters such as the mechanical shear strength and shear wave velocity V_s [4]. The dynamic compaction and vibro-compacting techniques are used by different companies. They have a wide field of application and are ideal for sandy-gravel fills [5, 6]. The objective of these techniques is to improve the soil performance, both in terms of settlements reduction, and of bearing capacity increase of the especially in seismic zones.

The shear velocity V_s is a significant parameter of the compaction quality. However, it needs to be correlated with in situ tests such as the CPT (Cone Penetration Test) [2]. These correlations between the parameters can be made if some experience already exists. Such work can permit to double the values of the shear mechanical strength parameters obtained by in situ tests [7]. As for shear waves, their value can be increased by more than 20% on the first meters [5].

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The design of foundations in a seismic zone consists initially to calculate the seismic forces applying to the foundation and secondly to check the resistance of the foundations to these efforts. In most building regulations, the structure of the response and loads arriving on the foundation are calculated neglecting the Soil-Structure Interaction (ISS) effects. Indeed, the latter is considered to have a favorable role in decreasing the inertial forces because they tend to lengthen the period of oscillation of the structure and thus shift the response to areas of lower spectral accelerations. This opens up a great debate about what are the important variables for ISS and further increases the need to develop methods able to include the soil-structure interaction in the analysis of all buildings [8]. The study of this phenomenon was particularly studied by many researchers like [9-12]. Some authors have proposed, for each degree of freedom of the foundation, soil models consisting of masses, springs and dampers with coefficients independent of the frequency. The simplest way to consider the soil is to represent it by springs connecting each other to a rigid base, to which one imposes a movement.

Several methods were proposed to solve the soil-structure interaction problem. To simplify the problem, linear analysis techniques were developed. One of the most commonly used approaches is the sub-structuring method that allows the problem to be analyzed in two parts. The system is separated into a superstructure and substructure. Each system can be studied separately [13-16]. The analysis of the foundation system can be reduced to compute the dynamic impedance functions and driving forces from incident waves [17, 18]. The impedance functions include the effects of the mass, stiffness, and energy dissipation properties of the subsoil as well as the radiation phenomenon (damping) [19].

Determining the dynamic response of the foundations and driving forces of the incident waves is a very complex process, especially when the geometrical configuration of the foundation and the soil support properties are complex. Several methods were introduced by several researchers to determine the dynamic response of foundations. The Boundary Integral Equation Method was used by Apsel (1979) [20]. These authors calculated the dynamic impedances and the foundations displacements. The boundary element method coupled with the finite element method was used by Padron et al (2007) to study the dynamic behavior of a pile group foundation. In the same context, the CIFEM method (Consistent Infinitesimal Finite Element Method) was used by Emanic and Maheshwari (2009) [21] to calculate the dynamic impedances of a group of 4 piles. The finite element method with absorbing boundaries was used by Messiod et al (2016b; 2017) [17, 22] to determine the dynamic response of an embankment and a slab placed on a reinforced soil by rigid piles. The response and behavior of machine foundations resting on dry and saturated sand was investigated experimentally by Fatah et al (2016) [23].

This work presents a three-dimensional finite element model with absorbing boundary to investigate the soil densification impact on the dynamic response of foundations. The MENARD Company based on a real case near Lyon (France), the soil densification is realized by dynamic compaction. It was considered by changing the soil class after the compaction work. Several soils at the class limit between D and C and between C and B are considered. These classes are presented in Eurocode 8 [24]. The dynamic response of circular and square foundations are analyzed at the limit of each soil class. The dynamic foundation response is calculated and presented in terms of the vertical and horizontal amplitudes for each soil category. The influence of the soil compaction, shape of the foundation and soil characteristic is analyzed in details.

2 Dynamic response of soil-foundation system

To obtain the response of the soil-structure system, the decomposition approach into a kinematic and inertial analyzes is applied using the superposition principle. The idea is to analyze the soil structure interaction problem in several successive steps, each of these steps being easier to solve than the global problem [13-16, 18]. The studied substructures are constituted by the soil and by the structure (Figure 1). The equilibrium equations of the soil-foundation system are written in a global form, and then the rigidity conditions at the interface soil-foundation are applied (continuity of displacements and stresses).

For the problem formulation, the external boundary must be placed sufficiently far from the structure to avoid the wave's reflection from the soil-structure interface during the analysis (Figure 1). In this case, this procedure is a very expensive one (due to the high number of degrees of freedom of the discrete system). To minimize the calculation time and to avoid the reflection of the waves at the boundary, the finite element method is used with adding absorbing boundaries. The equilibrium equation of the soil-structure system in the frequency domain is written as:

$$P = (-\omega^2 M + i\omega C + K)U \quad (1)$$

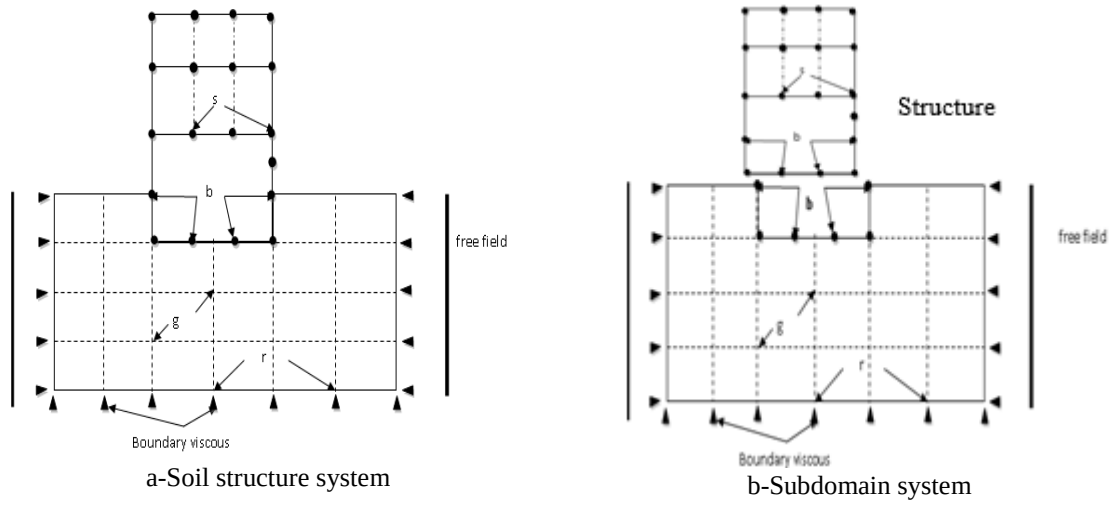


Fig. 1 - Soil structure interaction model

M and K respectively denote the mass and stiffness matrices of the system and C the damping ratio, U represents the displacement vector, and P is the load vector applied to the external boundary of the system. $U = U(\omega)$ is the complex amplitude of the response which is defined as the Fourier transform of $U(t)$:

$$U(\omega) = \int_{-\infty}^{+\infty} U(t) e^{-i\omega t} dt \text{ and } U(t) = \frac{1}{2\pi} \int_{-\infty}^{+\infty} U(\omega) e^{-i\omega t} d\omega \quad (2)$$

With $U = \begin{Bmatrix} U_s \\ U_b \\ U_i \\ U_r \end{Bmatrix};$

U_s : absolute displacements of the structure nodes; U_b : absolute displacements of the base nodes (Soil-Structure Interface); U_i : absolute displacements of the soil interior nodes; U_r : absolute displacements of the boundary nodes.

The considered excitation is a seismic one, therefore only the boundary nodes are loaded. The matrices blocks corresponding to the nodes of the soil-structure interface (common nodes s and g) are added. Either for the U_f is the vector of displacements of the free field boundary nodes, these displacements are not influenced by the presence of the structure when the boundary is placed far enough. In this case the following relation is considered:

$$U_r = U_r^f$$

By eliminating the degrees of freedom from the boundary of the model, the equation of motion of the soil-structure system becomes globally:

$$\begin{bmatrix} S_{SS} & S_{sb} \\ S_{bs} & S_{bb}^s + S_{bb}^s \\ & S_{ib} \\ & S_{ii} \end{bmatrix} \begin{Bmatrix} U_s \\ U_b \\ U_i \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ S_{ir} U_r^f \end{Bmatrix} \quad (3)$$

With $S = (-\omega^2 M + i\omega C + K)$ the complex dynamic stiffness matrix of the system.

By eliminating in equation (3), the degrees of freedom of the interior nodes (index i) using equation (3):

$$U_i = S_{ii}^{-1}(-S_{ir} U_r^f - S_{ib} U_b) \quad (4)$$

The second equation of (3) becomes:

$$S_{bs} + (S_{bb}^s + S_{bb}^g) U_b - S_{bi} S_{ii}^{-1}(-S_{ir} U_r^f - S_{ib} U_b) = 0 \quad (5)$$

The equilibrium system of the soil-foundation is given by equation (6)

$$S_{bs}U_s + (S_{bb}^s + S_{bb}^g - S_{bi}S_{ii}^{-1}S_{ib})U_b = S_{bi}S_{ii}^{-1}S_{ir}U_r^f \quad (6)$$

Substituting equation 6 into Equation 3, the soil-structure interaction response is obtained by the following form:

$$\begin{bmatrix} S_{ss} & S_{sb} \\ S_{bs} & S_{bb}^s + S_{bb}^g - S_{bi}S_{ii}^{-1}S_{ib} \end{bmatrix} \begin{Bmatrix} U_s \\ U_b \end{Bmatrix} = \begin{Bmatrix} 0 \\ S_{bi}S_{ii}^{-1}S_{ir}U_r^f \end{Bmatrix} \quad (7)$$

Assuming

$$\bar{S}_{bb}^g = S_{bb}^g - S_{bi}S_{ii}^{-1}S_{ib} \quad (8)$$

$$\bar{S}_{br} = S_{bi}S_{ii}^{-1}S_{ir}U_r^f \quad (9)$$

Finally, the formulation of soil-structure interaction is obtained by equation (10)

$$\begin{bmatrix} S_{ss} & S_{sb} \\ S_{bs} & S_{bb}^s + \bar{S}_{bb}^g \end{bmatrix} \begin{Bmatrix} U_s \\ U_b \end{Bmatrix} = \begin{Bmatrix} 0 \\ \bar{S}_{br}U_r^f \end{Bmatrix} \quad (10)$$

This equation is a possible formulation for the dynamic soil-structure interaction analysis of the expressed in absolute movements.

Equation 8 presents the dynamic stiffness matrix at the soil-foundation interface (the dynamic impedance functions). The determination of this matrix is the main point of this study. It can be determined by the application of vertical and horizontal forces at the foundation center.

$$[\bar{S}_{bb}^g]\{U_b\} = \{P_i\} \quad (11)$$

With: P_i the vector of forces (moments) coming from the inertial effect of the superstructure and U_b is the displacement vector at the center of the soil-foundation interface which is supposed to be rigid. The dynamic impedance functions can be obtained by following formula:

$$[\bar{S}_{bb}^g] = [\bar{S}_{bb}^r(\omega) + i\bar{S}_{bb}^{imag}(\omega)] \quad (12)$$

This expression presents a real part $\bar{S}_{bb}^r$ and an imaginary part $i\bar{S}_{bb}^{imag}$, when the mass of the foundation is not null, it is advisable to replace $[\bar{S}_{bb}^g]$ by $[\bar{S}_{bb}^g - \omega^2 M]$ in the equations above, with $[M]$ the mass matrix of the foundation.

Considering equation 11, the dynamic impedance functions are calculated after the determination of the resulting displacements at the application point of the harmonic force:

- the vertical dynamic impedance $S_V(\omega) = \frac{(P_0)e^{i\omega t}}{(U_z)(\omega)}$,
- the horizontal dynamic impedance $S_H(\omega) = \frac{(P_0)e^{i\omega t}}{(U_x)(\omega)}$,

With, P_0 is the amplitude of the vertical or horizontal forces applied to the foundation center, U_z and U_x the resulting vertical or horizontal displacements. The dynamic response of a massless foundation was obtained by the following expression [25]:

$$|u_0| = \left| \frac{F}{K_s [k(a_0) + ia_0 c(a_0)]} \right| \quad (13)$$

Where $|u_0|$ is the dynamic displacement amplitude, F is the force amplitude, K_s is the static stiffness, $k(a_0)$ is the stiffness coefficient, and $c(a_0)$ the damping coefficient with a_0 the dimensionless frequency. To simplify the analysis and

the comparison between the results of the numerical calculations and the experimental data, the ratio given by equation 14 is used, i.e., the amplitude of displacement is:

$$\left| \frac{1}{K(a_0)} \right| = \left| \frac{U_0(t)}{F_0(t)} \right| = \frac{1}{Ks\sqrt{K(a_0)^2 + a_0 C(a_0)^2}} = \frac{1}{F} \quad (14)$$

3 Calculation and validation model

This section examines the dynamic response of a rigid massless foundations placed on a viscoelastic soil limited by a rigid bedrock. The interacting elements considered using a finite element method (Code-Aster) [22]. In this validation part, a three-dimensional study of the dynamic response of a rigid, massless, square- foundation, subjected to the action of vertical and horizontal loads applied to the center of the foundation is presented. The surface foundation is placed on a soil consisting of two soil layers. The first one is a soft soil layer and the second is a hard soil layer.

3.1 Characteristics of model

Figure 2 shows the 3D model used for the numerical simulation. The height of the soft soil layer is equal to 3 m and the height of the hard soil layer to 5m. The foundation is represented by structural elements. The structural elements are perfectly linked to the volumic elements. The studied volume of soil has the following dimensions 40x40x8m. The geometric configuration and the nature of the applied load make it possible to apply symmetry conditions for the calculation of the dynamic impedance (only one half of the system is considered). The mechanical properties of the soil layers and foundations are presented in Table 1.

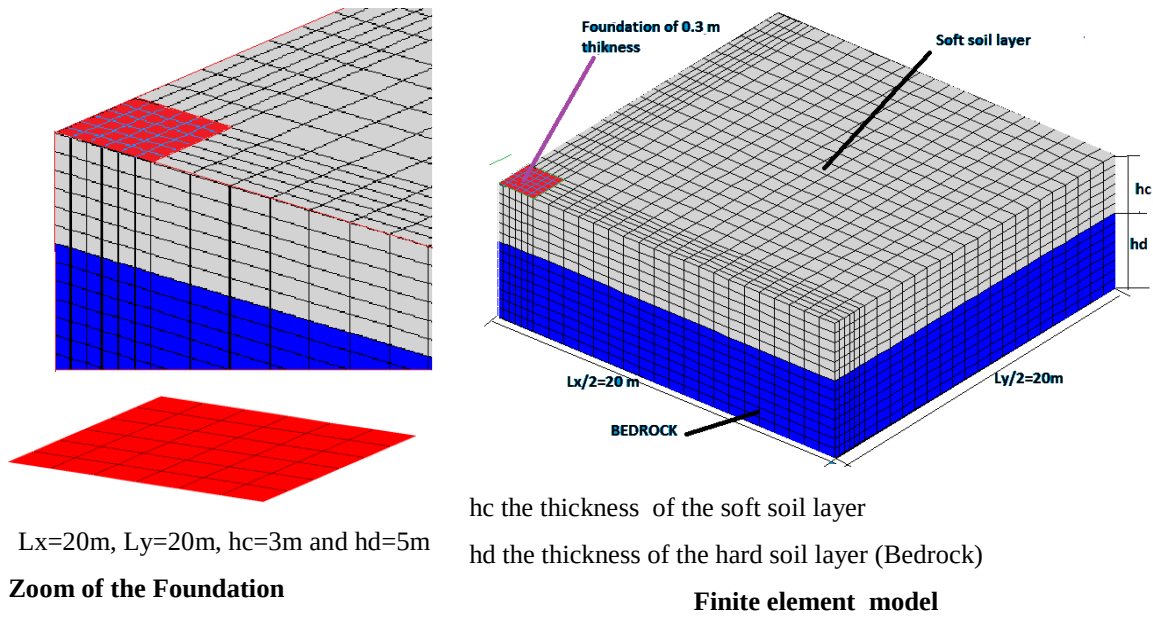


Figure. 2 - Quarter numerical model for the square foundation

The size of the mesh elements should be small enough like that the mesh can transmit the waves in an appropriate manner and without distortion of these waves. This size depends on the frequencies involved and of the wave propagation velocity [17, 22]. The loading frequencies and wave propagation velocity affect the accuracy of the numerical solution. [26] show that the element size should be smaller than 1/8th to 1/10 of the wavelength λ . As a result, the maximum frequency that can be modeled correctly for a mesh is given by:

$$f = \frac{C_s}{10 \cdot \Delta l} \quad \text{with} \quad f = \frac{\omega}{2\pi} \quad (14)$$

With; C_s the shear-wave velocity, Δl the size of the mesh element and ω the excitation frequency. The maximum frequency is depending on the maximum size of the element (0.5m is considered in this study).

In this section a comparison of the results obtained by the numerical calculations and the results obtained by BEM-TLM method [16, 18] is presented.

Table. 1- Mechanical characteristics of the constitutive materials

Parameters	Raft(Foundation)	Soft soil	Hard soil
Young's modulus (MPa)	30000	10	100
Density	2.5	1.8	2.0
Damping ratio	0.02	0.05	0.05
Poisson's ratio	0.25	0.3333	0.30

The numerical analysis was carried out on a square foundation without mass ($M = 0$) and of 1m of side. A vertical force of 1 MN was applied to the foundation center over a quarter of the model.

Used the BEM-TLM method [16, 18], the same geometry and parameters were used as for the numerical calculation.

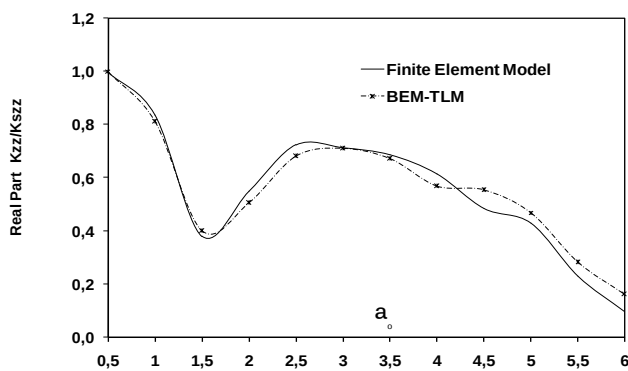


Fig. 3a - Vertical dynamic impedance K_{zz}

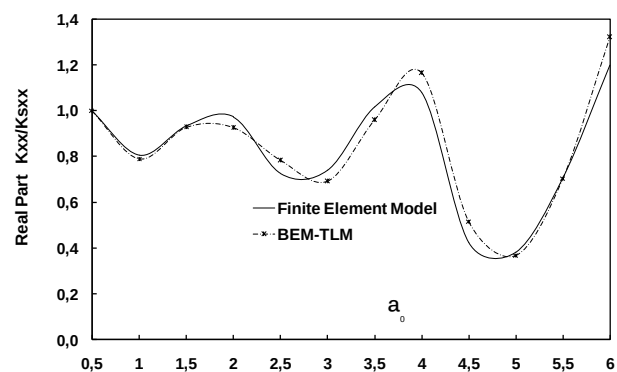


Fig. 3b - Horizontal dynamic impedance K_{xx}

The dynamic impedances are calculated by the compatibility condition. The obtained results in terms of stiffness are normalized with the static stiffness and are compared. Figure 3a shows the variation of the vertical dynamic impedance as a function of the adimensional frequency a_o . The impedance function was obtained by two different methods, the finite element method and the BEM-TLM method (Figure 3). The obtained results are in good agreement with those obtained by the BEM-TLM method up to the frequency $a_o = 1.5$. After this frequency, it can be noticed that there is a difference whose error percentage does not exceed 10%, which is acceptable. The horizontal dynamic impedance is calculated by applying a horizontal harmonic load of 1MN. Figure 3b shows that the dynamic translation impedance obtained by the BEM-TLM method is in good agreement with the one calculated by the finite element method. A slight difference was noted for some frequencies and especially for the dimensional frequency $a_o = 4$. This slight difference is due to the calculations algorithms used which are different for these two methods.

4 Soil densification by compaction

4.1 Modification of the shear velocity V_s by soil densification

The soil densification permits to improve certain parameters such as the bearing capacity q_c and the shear waves velocity V_s . Brûlé and Javelaud(2012) [7] have experimentally shown that the values of the mechanical parameters can be doubled by this process. As for the shear waves, their value can be increased by more than 20% on the first meters [5]. Considering the velocity of the waves at 30m depth $V_{s,30}$, it permits to classify soils according to Eurocode 8 [24] as follows:

$$V_{s,30} = \frac{30}{\sum_i \frac{h_i}{v_i}} \quad (15)$$

The velocity intervals serving to classify the soils according to Eurocode 8 [24] are large A, B, C, D and E, compared to the gain brought by compaction. Thus, a soil class change is conceivable if the initial velocity V_s is near the upper bound of the interval. It may be interesting to study $V_{s,80}$ to determine the effect of compaction at great depths:

$$V_{s,80} = \frac{80}{\sum_i \frac{n_i h_i}{v_i}} \quad (16)$$

To study the compaction effect, a soil class change is assumed after the compaction work Lee and Gu (2004) [27]. Two cases were successively be considered: at the limit of class D and C and at the limit of class C and B. Circular and square foundations are placed and analyzed at the limit of each of these two soil cases.

V_s is a significant parameter of the compaction quality. This parameter can be correlated with in situ tests such as the CPT [2]. These correlations between the parameters should be carefully made. In order to propose a first comparative case, the following study is based on a real case near Lyon (France). The lithology of the site consists of a layer of a heterogeneous embankment of 15m height, overlying a sandy-gravel alluvium more than 30m thick. The whole rests on a granitic and/or metamorphic substratum. However, in order to compare the chosen methods, the soil is considered to be a monolayer. A set of tests was realized and collected by (Brûlé et al., 2010a) [4].

The compaction operation was done in two phases. During the mass impact, the energy is transmitted to the soil, and P and S waves are developed. The compression waves P liquefy the soil, and then the shear waves S shear the soil grains. Surface waves will then appear that represent about 70% of the propagated energy. This results in a lowering of the platform level due to settlements. It then permits to improve the mechanical properties and homogeneity of the granular soils [2, 28]. In order to analyze the influence of the modification of V_s according to the type of soil class, a constant gain of 3.4% between the shear wave velocity before and after compaction is considered. The results and parameters of the class changes used are presented in Table 2.

Table. 2 - Categories of considered densified soils [4])

	Class	$\gamma(\text{kN/m}^3)$	$V_{s,30}(\frac{m}{s})$
Before	D	17	177
After	C	18	183
	Class	$\gamma(\text{kN/m}^3)$	$V_{s,30}(\frac{m}{s})$
Before	C	18	353
After	B	19	365

5 Effect of the soil densification on the dynamic response of foundations

In order to analyze the effect of soil densification on the dynamic behavior of foundations, a series of rigid massless foundations is considered in this analysis. Circular and square massless foundations are studied in order to optimize the foundation geometry. The foundations are placed on the compacted soil at the end of each compaction phase. The influence of the soil densification by compaction is studied by assuming a change of soil class after the compaction operation.

5.1 Circular Foundations

Figure 4 shows the 3D finite element model used to study the effect of soil densification on the dynamic response of the foundations. The numerical simulations are realized using the ASTER® code. The considered dimensions of the foundations radiuses are 0.5m, 1m, 2.5m and 5m. The geometric characteristics of the model were determined using a parametric study to ensure the correct functioning of the absorbing boundaries. The studied volume of soil has of the dimensions 40 x 40 x 15 m³. The foundation is in contact with the soil and supposed infinitely rigid (0.2 m of thickness). The elements of the model are considered viscoelastic materials. The mechanical properties of the densified soil layers are presented in Table 2.

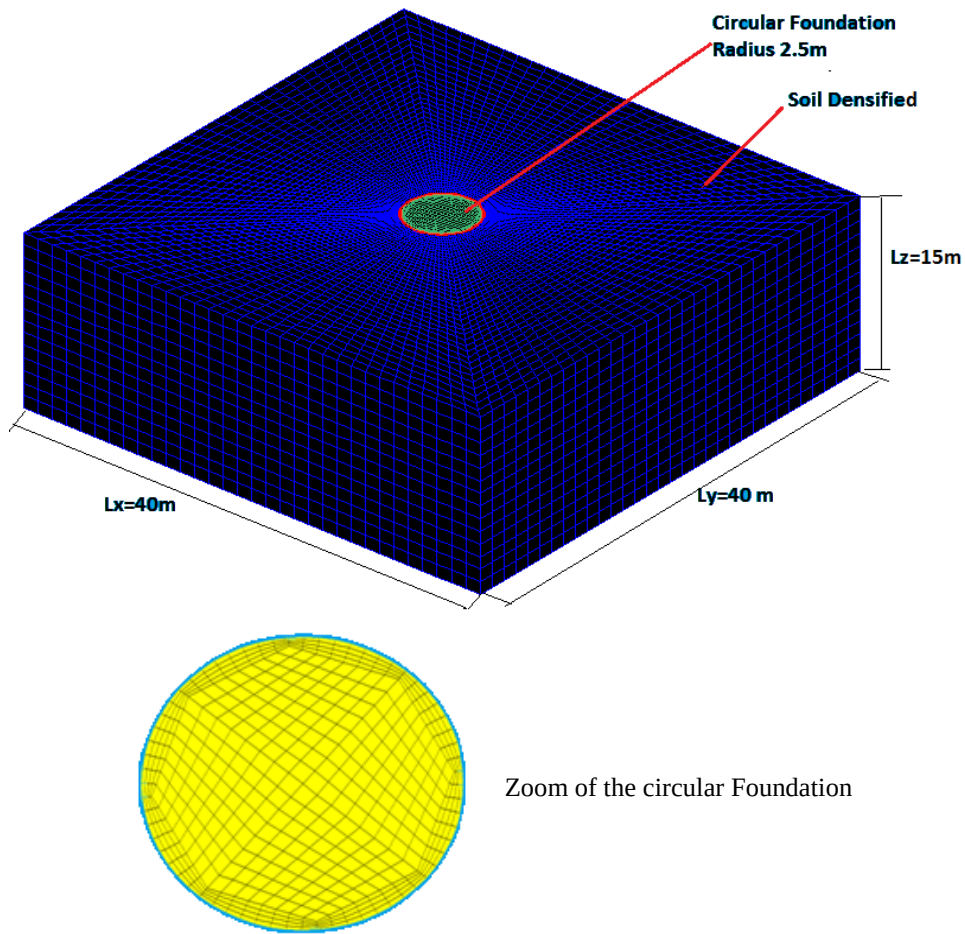


Fig. 4 - Numerical model

The results of a serie of massless circular foundations were analyzed in order to better understand the behaviour of the soil-foundation system before and after the compaction operation. Figures 5 to 8 show the effect of the soil densification on the variation of the amplitudes (vertical and horizontal displacements) as a function of the excitation frequency. These figures show that the amplitudes of the displacements are considerably affected by the effect of the soil densification, especially for the low frequencies. This affectation is caused by the increase of the shear wave velocity V_s . The increase of the vertical and horizontal displacements is inversely proportional to the phases of densification (soil hardening), the amplitudes of the displacements are reduced according to the increase of the soil characteristics. The soil densification increases the shear modulus G , shear wave velocity V_s and soil density, while decreasing the amplitudes of displacements.

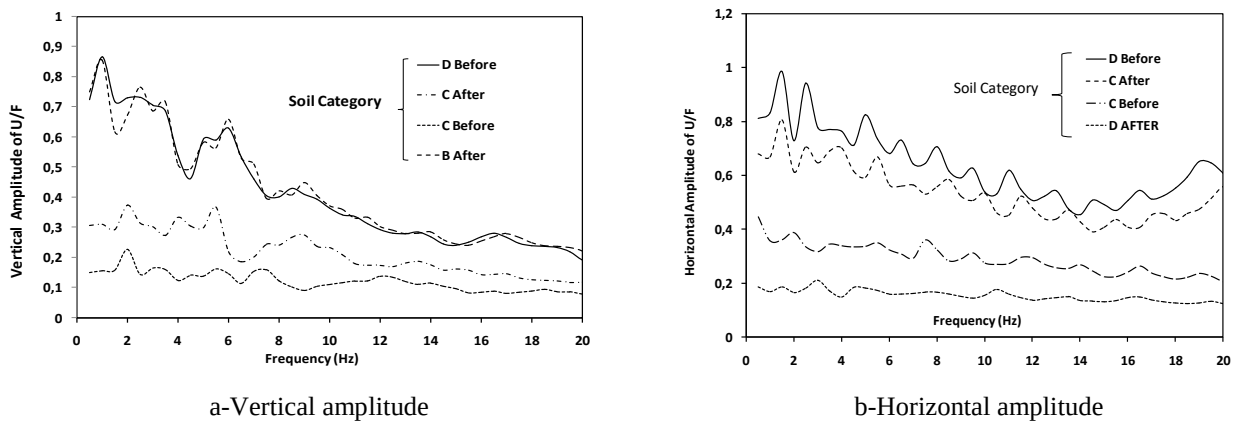


Fig. 5 - Influence of the soil densification on the amplitude of displacement $|U/F|$, $R=1m$

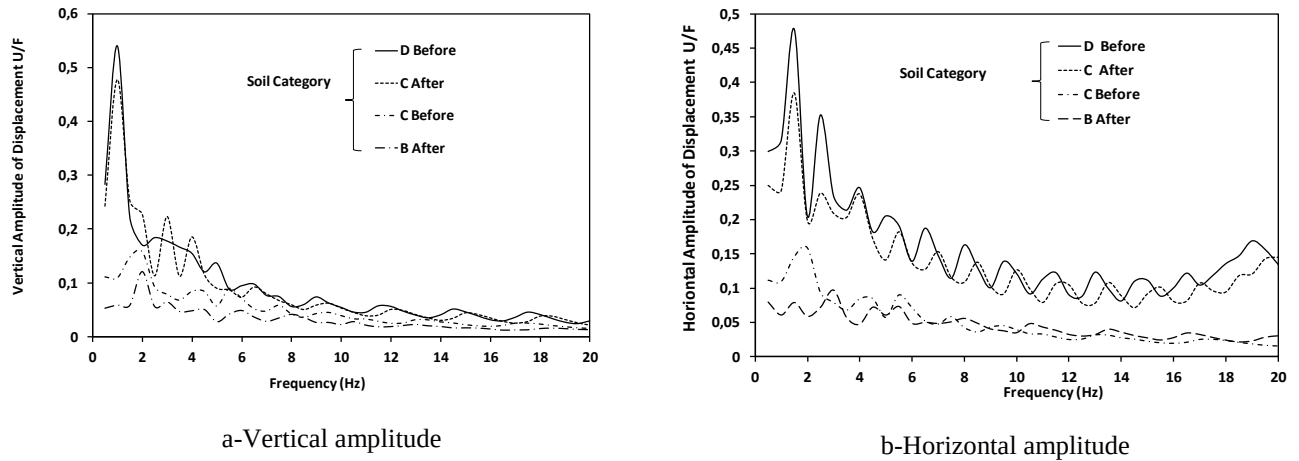


Fig. 6 - Influence of the soil densification on the amplitude of displacement $|U/F|$, $R=2.5m$

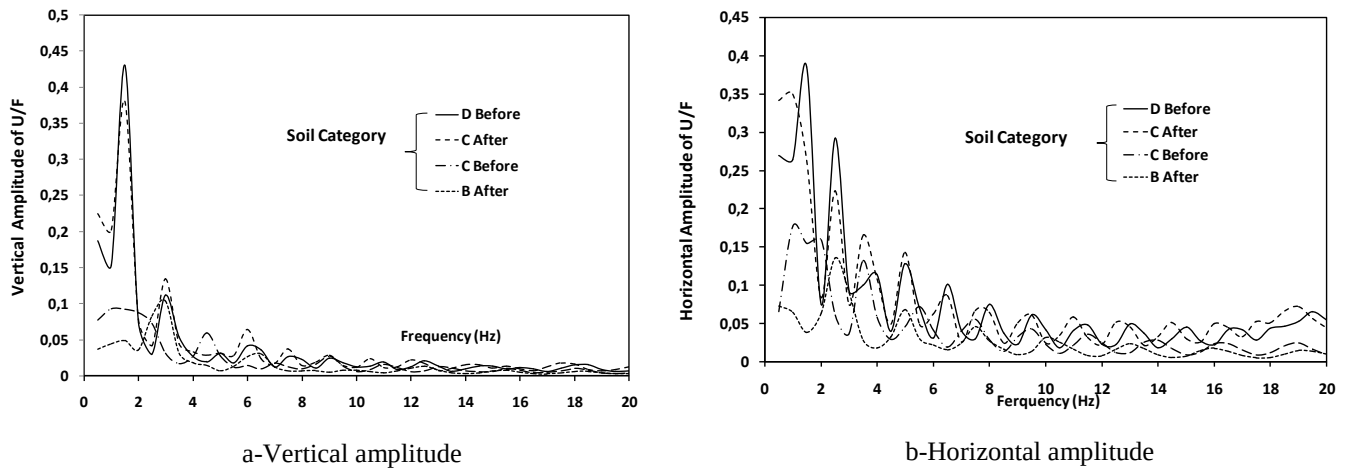


Fig. 7 - Influence of the soil densification on the amplitude of displacement $|U/F|$, $R=5 m$

Figures 5 to 7 show that the decrease rate of the amplitudes of displacement for the four radiuses is fairly low for the case where the soil class is D before the compaction and C after. On the other hand, this decrease rate is faster when the soil class passes from class C before to B after. Finally, the obtained amplitudes of the displacements are mainly affected by the soil densification especially for low frequencies and for all the foundation cases.

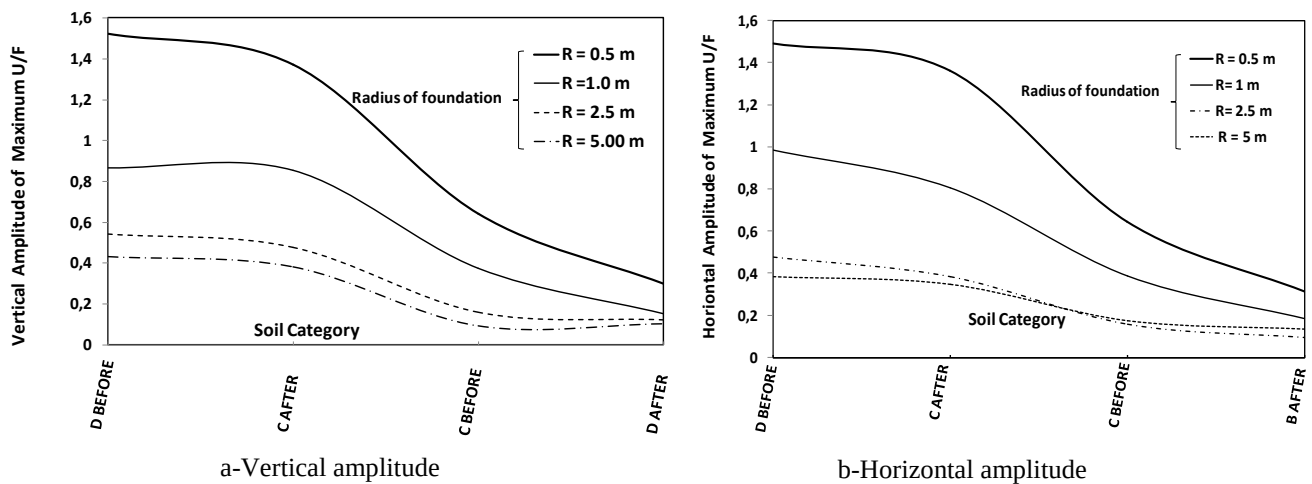


Fig. 8 - Variation of the amplitude of the maximum displacements as a function of the compaction phases

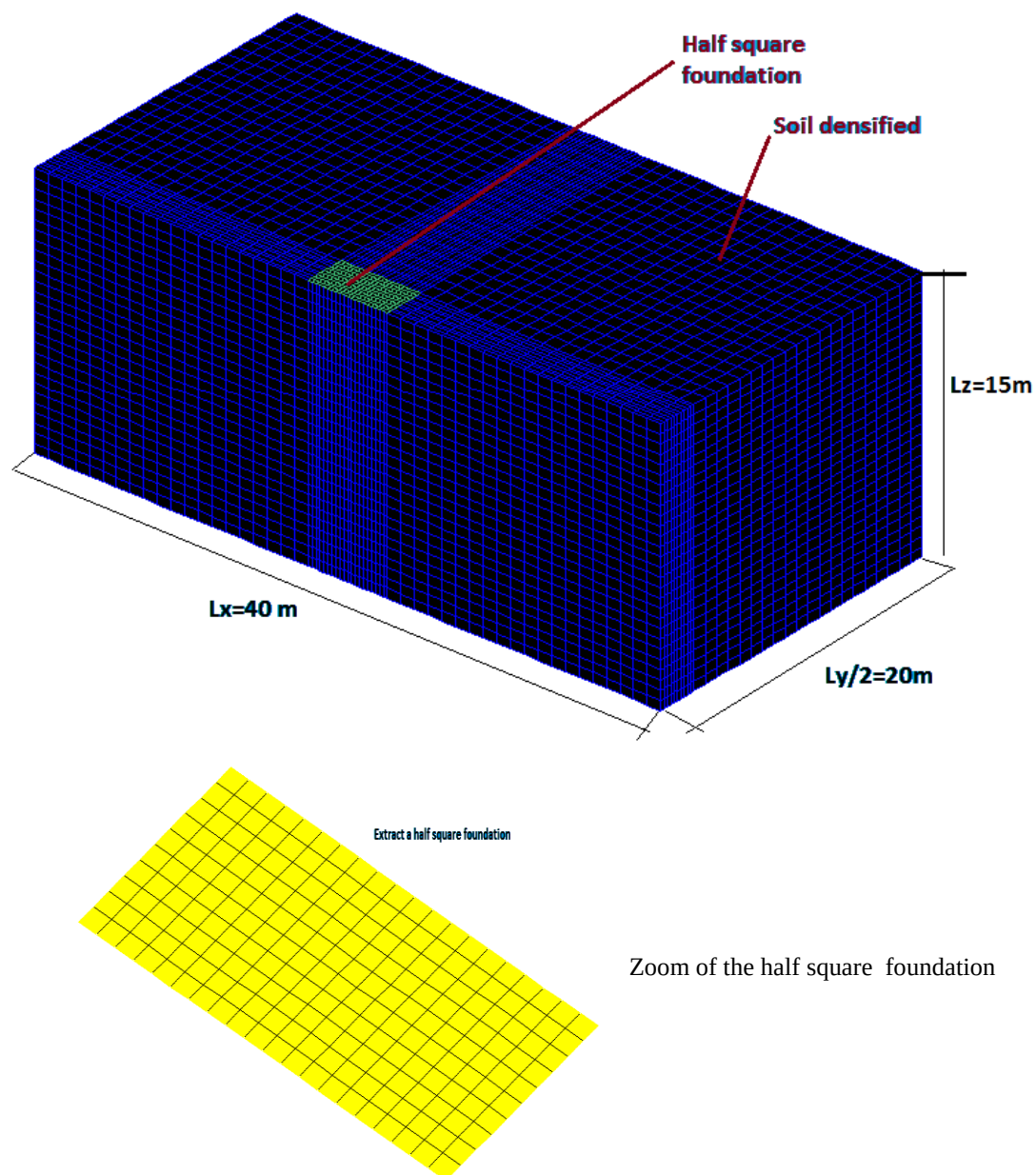


Fig. 9 - Half numerical model adopted

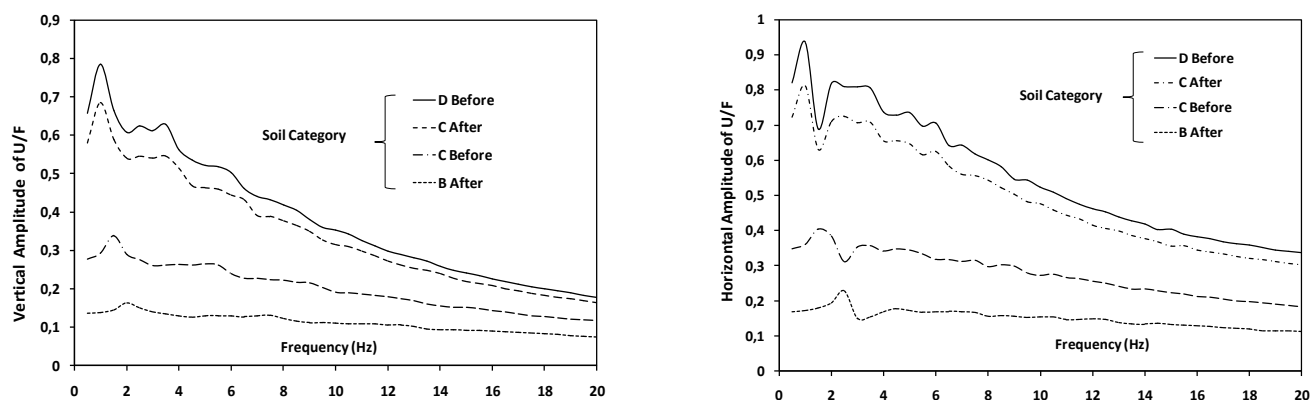


Fig. 10 - Influence of the soil densification on the displacements amplitude $|U/F|$ for $B=0.885\text{ m}$

Figure 8 shows respectively the soil densification effect on the maximum amplitudes of vertical and horizontal displacements. The obtained results show that the maximal displacements are strongly affected by the compaction operation. The maximum settlements and translations are given for the soil D before the compaction operation. The obtained results also show that the amplitudes of displacements decreased as a function of the increase of the soil-foundation surface contact. For all the soils categories and foundations radiuses, the amplitudes of the displacements are reduced considerably according to the soil densification phases (Figure 8).

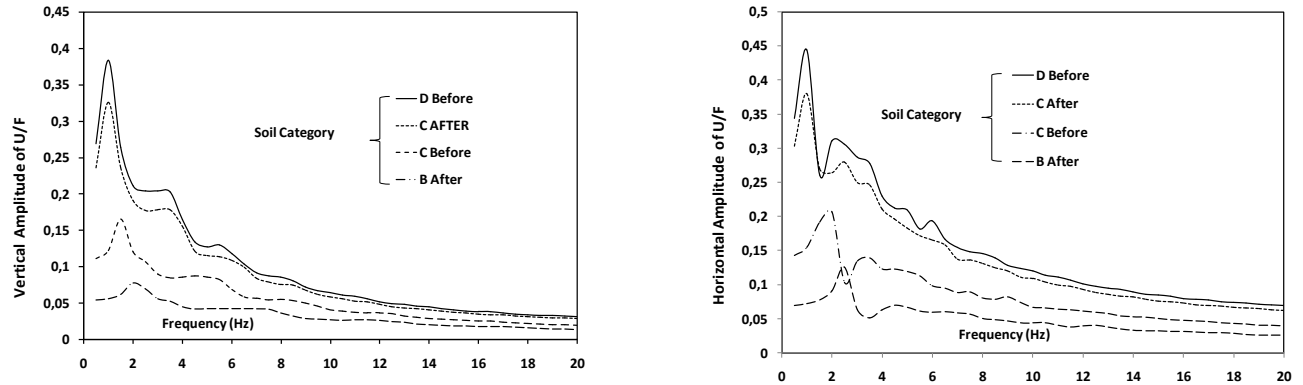


Fig. 11 - Influence of the soil densification on the amplitude of displacement $|U/F|$ for $B=2.225m$

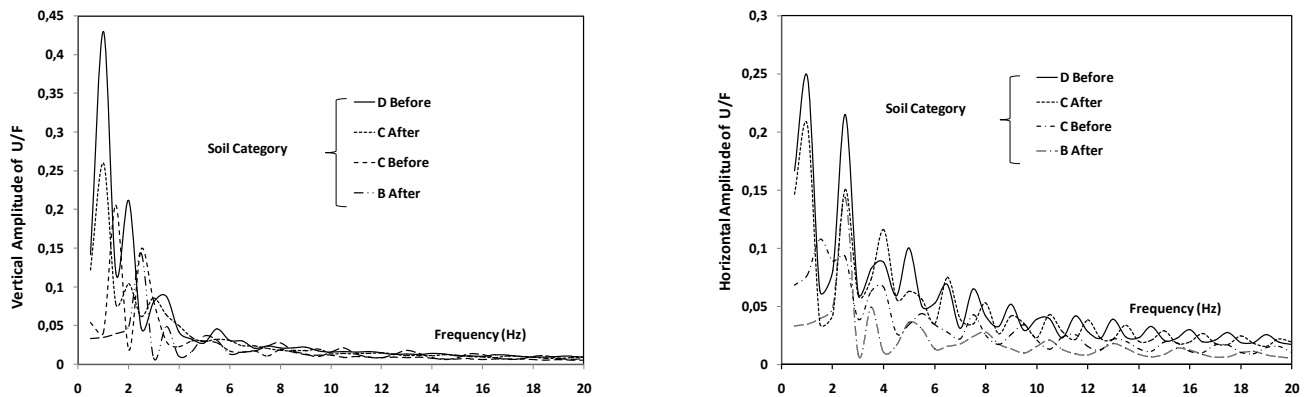


Fig. 12 - Influence of the soil densification on the amplitude of displacement $|U/F|$ for $B=4.3m$

5.2 Square massless foundations

In this section a series of square massless foundations is analyzed. Circular foundations of the preceding section are replaced by square foundations, the dimensions of these foundations are determined using an equivalent surface (the square foundation have the same surface as the circular one $(B \times B) = A = \pi r^2$). Taking the radius of the circular foundations respectively equal to 0.5m, 1m, 2.5m and 5m, the equivalent dimensions for the squared ones are $B=0.45m$, $0.885m$, $2.225m$ and $4.3m$. Using quarter and half numerical models for the dynamic analysis, the obtained results are shown in Figures 9 to 11

The obtained results are shown in Figures 10-12; the evolution of the amplitudes of displacements is the same as for the circular-shaped ones. The impact of soil the densification on the dynamic response is important. The amplitudes of the displacements are reduced when soils pass from one category to another, but the values of the amplitudes are different to those of the circular foundations. The obtained results show that the compaction is an adequate method of soil reinforcement that allows reducing the displacements.

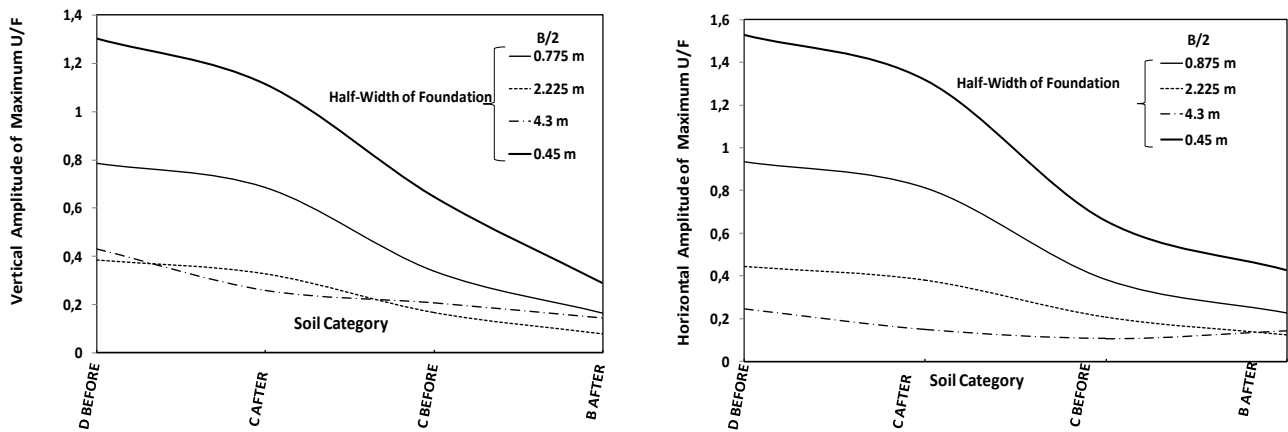


Fig. 13 - Variation of the maximum amplitudes of the displacements as a function of the compaction phases

The results show that the square shape foundation seems to be the best solution especially for large raft foundations (Figure 13). The amplitude of vertical displacements is reduced compared to the circular shape foundation one (Figure 13).

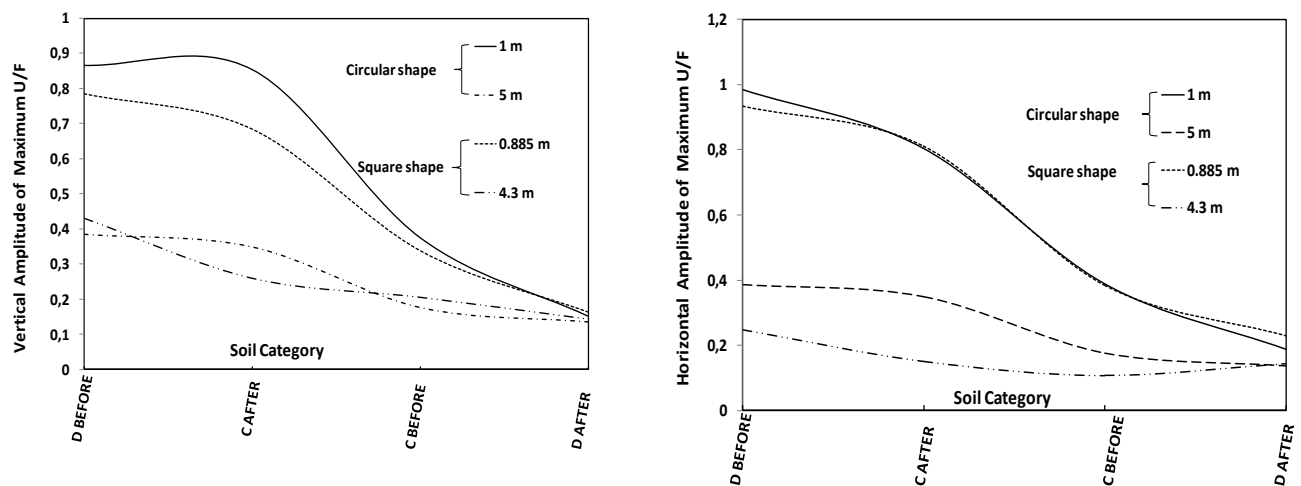


Figure. 14-Variation of the maximum amplitudes of the displacements for the square and circular foundation

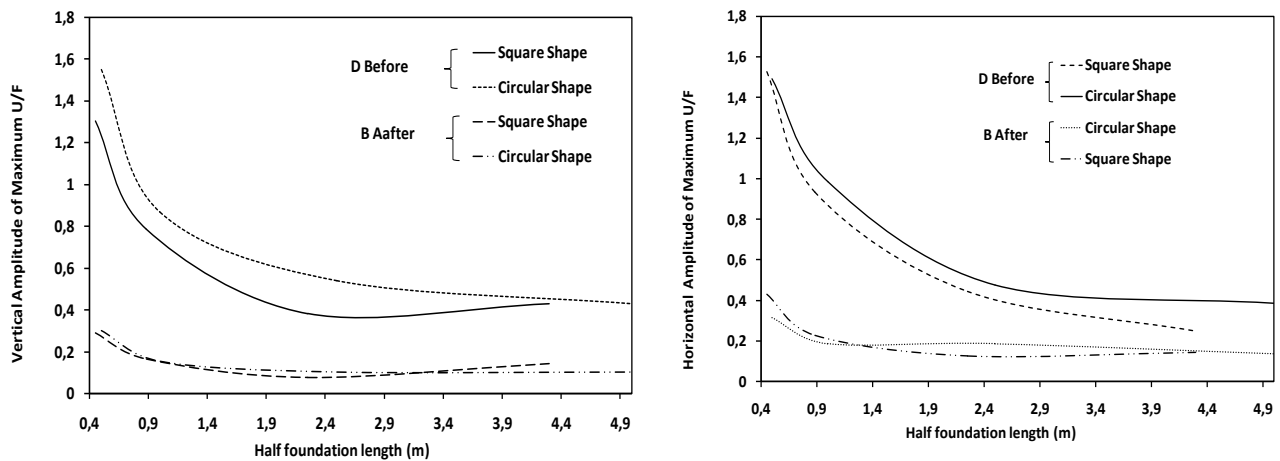


Fig. 15 - Influence of the shape of the foundation on the dynamic response

The results show that the increase of the amplitudes of the displacements is inversely proportional to the increase of the foundations dimension. The amplitudes of the displacements are affected by the increase of the foundation surface. The amplitudes of the displacements are reduced in a considerable way with the increase of the foundations dimension (Figures 14 and 15).

The influence of the foundation shape on the dynamic response is presented in figure 15. The results show that the vertical and horizontal displacements of the square foundations are inferior to those of the circular ones.

6 Conclusion

In this work a 3D finite element numerical model is proposed to study the influence of the soil densification on the dynamic response of circular and square foundations. The soil densification was done by dynamic compacting operation in a real France site (near Lyon (France)). To study the influence of the soil densification, a change of the soil class is assumed after the compacting operation. In order to analyze the influence of the modification of V_s according to the type of class, a gain of 3.4% for the shear wave velocity for each class of soil before and after compacting operation is considered. The sub structure method was used for evaluating the dynamic response foundations. The vibrations of the systems have been studied by the principle of causality, by applying a vertical and horizontal load to the centers of the foundations. The obtained results are presented in terms of amplitudes of the displacements in the translations mode.

In this study the results of a series of massless circular and square foundations were analyzed in order to better understand the behaviour of the soil-foundation system before and after the compaction operation. The soil compaction is an adequate method of soil reinforcement. It then permits to improve the mechanical properties of soil especially, increases the shear modulus G , shear wave velocity V_s and soil density, and that allows reducing the displacements. . The amplitudes of the displacements are considerably affected by the effect of the soil densification, especially for the low frequencies. The reduction percentage of displacements for the soils categories after compaction is approximately 10% lower than the one before compaction. The fundamental frequency is changed after the soil densification operation between 1 Hz and 1.5 Hz. The influence of the foundation size on the dynamic response is an important parameter; the amplitudes of the displacements are reduced in a considerable way with the increase of the foundations dimension and the amplitudes of displacements of the square foundations are inferior to those of the circular ones.

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