A NEW CALCULATION METHOD FOR THE BEARING CAPACITY OF SHALLOW FOUNDATIONS FROM THE SHEAR WAVES VELOCITY

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ABSTRACT
A new method of designing shallow foundations has been proposed in this article. It consists of using the propagation velocities of the seismic waves Vs, obtained in geophysical prospection on the construction sites of structures. The validation of this method has been done through geotechnical reconnaissance campaigns, supplemented by investigations aimed at obtaining the geotechnical parameters of dynamic type, in particular the soil modules with low deformations. Two cross-hole surveys were conducted at two sites, as well as 2 SPT surveys and 2 pressure meter surveys. These investigations were conducted on the same boreholes at the same time under perfectly controlled conditions. The mechanical properties allowing to characterize the deformability of the materials in “static” or with very small deformations were evaluated on the points of survey 1 and 2. From the cross-hole tests, the maximum shear modules of the grounds $G_{\text{max}}$ were deduced as a function of the propagation velocity of the shear waves $V_s$ and the density $\rho$ of the soils. The dynamic Young's modulus of these soils $E_{\text{dy}}$ was then determined as a function of $G_{\text{max}}$, after which a correlation was established between this dynamic Young's modulus and the undrained shear strength $c_u$ of the soils. From $c_u$ the foundation has been designed by the traditional calculation method (in $c$ and $\varphi$) which has the merit of being known in all countries. The results obtained were compared to the calculation of foundations based on the in-situ tests commonly used for the design of foundations of structures (Standard Penetration Test and the pressure meter test). The difference between the proposed design method $V_s$ and the pressure meter and SPT methods in terms of bearing capacity $q_u$ is low, of the order of -6.78% and -0.28% for the SPT method and 25.25% for the pressure meter method.
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**Key words:** Design of shallow foundation, seismic wave velocity $V_s$, cross hole survey, SPT survey, pressure meter survey, bearing capacity $q_u$.


1. **INTRODUCTION**

Shallow foundations are considered as simple and easily executed structures. Many studies have been devoted to them for nearly a century to establish, validate and improve their methods of calculation, because the parameters to be taken into account are varied and these calculations involve many facets of the mechanical behavior of soils. Traditional calculation methods (in $c$ and $\phi$) have the merit of being known in all countries, sometimes with local variants; they use the shear strength characteristics, which are essential in other areas of soil mechanics that predominantly determine the behavior of soil at failure. They also have the advantage of using the same parameters as retaining walls calculations or slope stability calculations and to extend naturally in numerical calculation methods.

In this article, we develop a new concept, based on geophysical seismic survey data, from the Cross-hole test. Information obtained allows us to design shallow foundations by the traditional method (in $c$ and $\phi$). The results are compared to the calculation of foundations based on two in-situ tests commonly used for the design of foundations (SPT and pressure meter test). The three tests (SPT, pressure meter, cross-hole) were carried out on two sites on the same boreholes and at the same time, under perfectly controlled conditions [1-3], to validate the results of calculations from cross-hole geophysical survey data. Two phases of ground investigation have been carried out at the site and identified a Lateritic Residual Soil in the shallow deposits. The soil is described as a firm to stiff, becoming stiff and very stiff clay with increasing depth. In accordance of EC8 BS 1998-1-2004 + A1:2013 [3] the principal Residual Soil, soil type at site is classified C and B.

2. **METHODOLOGY**

The cross-hole test gives us possibilities to obtain, on a surveyed site, the propagation velocities of the primary wave $V_p$ and secondary wave $V_s$. These wave velocities provide useful and exploitable information in geotechnic and civil engineering. In the field of small deformations, in the order of $10^{-5}$, the cross-hole tests permit us to deduce the maximum shear modulus of the soils $G_{max}$ from the propagation velocity of the shear waves $V_s$ and the density $\rho$ of the soils. There is likewise a relation linking $G_{max}$ to the Young's modulus $E$. From the Young's modulus $E$, the undrained shear strength $c_u$ of the soils is determined. It should be noted that currently, all the methods for calculating the impedances (stiffness and damping) of the foundations uses the geophysical parameter $V_s$ in order to evaluate the Soil Structure Interaction.

2.1. Importance of using Geophysical Data for the Design of Foundations

Geophysics and geotechnics are two disciplines related to soil studies. They rely on recognition techniques of their properties. These approaches to soil exploration are mainly based, for geotechnics on the recognition of the mechanical properties of soils and for geophysics, on the study of the physical properties of soils. Geophysics is interested in measuring the physical properties of soils (electrical resistivity, seismic velocity, ...). Geophysical measurements are called "indirect measurements" by the geotechnician. An
indirect measure means that the quantity measured is an indicator of the property sought. Indeed, the justification of a geophysical measurement is rarely the measure in itself; its main interest is, very generally, the description of the distribution of the measured physical property [4-5], in vertical profile (electrical survey) or horizontal (seismic refraction), or in horizontal cartography (gravimetry, magnetism, ... ) or vertical (seismic reflection section, electrical tomography) (for example, once inverted, the apparent resistivity can be used to determine the lithological nature of a formation). Each type of geophysical survey uses a volume of soil whose size is related to the dimensions of the measuring device. These volumes can range from several dm$^3$ (Prospecting EM) to several million of m$^3$ (Electricity Resistivity Tomography) depending on the chosen method, its mode of usage. The geophysical measurement obtained is conditioned by the set of properties present in the investigation volume. Thus, the measurement obtained is an average of the physical property studied within this heterogeneous volume. At this present state, the link between geophysical measurements and geotechnical parameters is not direct. It is generally necessary to cross geophysical results with other sources of information (geological maps, surveys, drilling, ...), to obtain information useful to the geotechnician (nature of layers, thickness, ...). Calibrations in the laboratory can at least be performed also to establish the existence of a relationship between the measured geophysical parameter and the desired property. Geotechnical reconnaissance makes it possible to evaluate the mechanical properties of soils. This evaluation aims to design a structure according to the mechanical behavior of the soil. This soil recognition permits to directly estimate the quantities used in soil mechanics (thickness, limit pressure, $N_{SPT}$, Young’s modulus, Poisson's ratio, etc.) that will be used for designing structures (foundations, retaining walls, embankment, dikes, ...). During geotechnical measurements, the mechanical properties are always evaluated punctually, at a point or on a set of points distributed along a vertical (sample, survey). The sampling interval (distance between two consecutive measurements along a profile) depends on the technique. It can range from centimeter (dynamic penetrometer) to meter (pressure meter). Geotechnical tests being relatively heavy and expensive on one hand, destructive on the other hand, they can only be conducted in a limited number of points. Extrapolation of geotechnical measurements, to an entire geological layer and its lateral extension, implicitly assumes hypotheses concerning the homogeneity of the geological layer and the distribution of mechanical properties within the layer. For reasons of cost, geotechnical engineering cannot determine and quantify the mechanical properties with a mesh of the order of m$^2$ on a parcel of several hectares. To achieve such a result, the number of tests to be performed would be considerable (10000 / hectare for example). The representability of geotechnical measurements in soil recognition is therefore extremely uncertain: it is impossible to obtain a geotechnical parameter distribution with a resolution and satisfactory accuracy.

The joint use of geophysical and geotechnical measurements makes it possible to respond to this problem of representativeness of the measurements and spatial location of the latter. Indeed, a geophysical campaign, properly conducted in relation to the objectives of the project can cover the entire surface and obtain an objective indicator in every point. Geophysics permits us to describe the distribution of soil heterogeneities by measurements of fields of physical properties. The combination of this source of information with point-in-time mechanical surveys permit to extend the geotechnical measurements to the surface or volume considered. This approach permits to answer the problem of the representability of the geotechnical measurements. By coupling, for example, static penetrometer (CPT) soundings with electrical resistivity tomography (ERT), it is possible to extend the information obtained from the surveys to areas with the same physical properties (Figure 1). Geophysics is used to characterize the scale of variability and to propose a structuring of the soil based on the
distribution of the measured physical property. It is possible to apply the data fusion to the 3D recognition of a basement. This 3D modeling makes it possible to propose a distribution of the physical and mechanical properties at any point of the considered volume, including in the zones without measurement.

![Coupling static penetrometer test (CPT) soundings with electrical resistivity tomography (ERT)](image)

**Figure 1** Coupling static penetrometer test (CPT) soundings with electrical resistivity tomography (ERT)

### 2.2. Basic Concepts of Shallow Foundation Bearing Load Calculations

The development of soil capacity prediction methods based on laboratory test results [6], i.e. using the Mohr-Coulomb failure criterion ($\tau_{\text{max}} = c + \sigma \tan \phi$), is already old and it becomes complicated to make an exhaustive description. Reference is made to the inventories prepared by various authors [7-12], who give a detailed account of the calculation of shallow foundations, as well as the Terzaghi and Peck’s manual [13] and those of Vesić [14] and Das [15]. Terzaghi and Peck [13] give indications on the theories developed and used until today.

The formula for calculating the bearing capacity of the shallow foundations given in Eurocode 7 (NF P 94-261) [16] has three terms, each comprising a bearing capacity factor and correction coefficients. For undrained calculations ($c_u$, $\phi_u$) the suggested formula is given by Equation 1.

\[
q_{\text{net}} = (\pi + 2)c_u b_c s_c \alpha + q
\]  

With:
- $q$, overload pressure at the base of the foundation,
- $b_c$, coefficient of inclination of the base:
  \[
b_c = 1 - \frac{2 \alpha}{\pi + 2}
\]
- $\alpha$, the inclination of the base of the foundation relative to the horizontal,
- $s_c$, form factor:
  \[
s_c = 1 + 0.2 \frac{B'}{L'}
\]
- $B'$, effective width of the sole,
- $L'$, The effective length of the sole,
Ic, coefficient of inclination of the load:

\[ i_c = \frac{1}{2} \left( 1 + \sqrt{1 - \frac{H}{A'c_n}} \right), \text{ with } H \leq A'. \]

\( H \), design value of the force parallel to the plane of the base of the sole.

From the results of the Standard Penetration Test, the bearing capacity of the foundation is determined from the following relation described by equations 2 and 3, proposed by Meyerhof [17].

\[ N_m = \frac{1}{2B - 0.5B} \int_{D+0.5B}^{D+2B} N(z) \, dz \]  

\[ q_u = \frac{3.0}{0.08} \left( 1 + \frac{D}{3.5B} \right) \left( B + 0.3 \right)^2 \text{ for } B \geq 1.2 \text{ m} \]  

The settlement of the foundation from the SPT data is evaluated according to the method of Burland and Burbidge [18], from the following equation given in equation 4.

\[ \text{if } q_{ref} \geq \sigma'_{v_0}, \quad s_f = \frac{1.71B^{0.7}}{N_m^{1.4}} \cdot f_1 \cdot f_2 \cdot f_3 \left( q_{ref} - \frac{2}{3} \sigma'_{v_0} \right) \]  

With:

- \( s_f \) in mm; \( B \) in m; \( \sigma'_{v_0} \) in kPa and \( q_{ref} \) in kPa.

Where the coefficients \( f_1, f_2, f_3 \) are respectively:

\[ f_1 = \left( \frac{1.25 \frac{L}{B}}{\frac{L}{B} + 0.25} \right)^2 ; \]  

\( f_2 = 1 \text{ for } D / B < 3, \text{ if not, } f_2 = 1.5 ; \) \( f_3 = 1.3 + 0.2 \log_{10} \left( \frac{t}{3} \right) \text{ with } t, \text{ the time considered in years.} \)

From the results of pressure meter test surveys, the bearing capacity of the shallow foundation is determined from the following relation, given by equations 5 and 6, taken from Fascicle 62 - Tire V [19].

\[ q'_u = k_p \cdot p_{le}^* - q'_0 \]  

\[ \log(p_{le}^*) = \frac{1}{1.5B} \int_B^{D+1.5B} \log(p_l^*(z)) \, dz \]

Where

- \( p_{le}^* \) is the equivalent net limit pressure,
- \( k_p \) is the bearing factor, determined according to the type of soil,
- \( B \) is the width of the base of the foundation,
- \( p_l^*(z) \), measured values, interpolated linearly on a logarithmic representation,

The final settlement \( s_f \) is calculated as follows by relation 7.
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\[ s_f = s_c + s_d \]  \hspace{1cm} (7)

\[ s_c = \frac{\alpha}{9E_c} (q'_{ref} - \sigma'_{v0}) \lambda_c B \]

\[ s_d = \frac{2}{9E_d} (q'_{ref} - \sigma'_{v0}) 0.6 \left( \frac{\lambda_d B}{0.6} \right)^{\alpha} \]

With \( \alpha \) is the rheological coefficient depending on the type of soil, \( \sigma'_{v0} \) is the effective vertical stress before works, and \( \lambda_c \) and \( \lambda_d \) are shape coefficients of the footing.

2.3. Calculation Method Proposed from the Seismic Wave Velocity \( V_s \)

In the field of small deformations, of the order of \( 10^{-5} \), the cross-hole tests make it possible to deduce the maximum shear modulus of the soils \( G_{\text{max}} \) (Figure 2) [20] from the propagation velocity of the shear waves \( V_s \) and the density \( \rho \) of the soils, according to equations 8 and 9.

\[ G_{\text{max}} = \rho V_s^2 \]

\[ E_{\text{max}} = 2(1 + \nu)G_{\text{max}} = 2(1 + \nu)\rho V_s^2 \]

Where \( \alpha \) is the rheological coefficient depending on the type of soil, \( \sigma'_{v0} \) is the effective vertical stress before works, and \( \lambda_c \) and \( \lambda_d \) are shape coefficients of the footing.

![Figure 2 Shear modulus ratio vs strain level (Durand, 2009)](http://www.iaeme.com/IJCIET/index.asp)

The dynamic Young modulus \( E_{\text{max}} \) is more important than the Young modulus in static [20-23]. In this study, two phases of ground investigation have been carried out at the site and identified a Lateritic Residual Soil in the shallow deposits. The soil is described as a firm to stiff, becoming stiff and very stiff clay with increasing depth. In accordance of EC8 BS 1998-1-2004 + A1:2013 [3] the principal Residual Soil, soil type at site is classified C and B. A correlation was found experimentally in the field of small deformations between the maximum Young modulus \( E_{\text{max}} \) and the undrained shear strength \( c_u \).

\[ E_{\text{max}} = 2(1 + \nu)\rho V_s^2 = 2500c_u \]

from where, \( c_u = \frac{(1 + \nu)\rho V_s^2}{1250} \)  \hspace{1cm} (10)

The bearing capacity of the shallow foundation is then determined from the following relation, given by equations 11 and 12, as appropriate:
The immediate settlement is given by equation 13.

\[
q_u = (\pi + 2) \left( \frac{(1 + \nu) \rho V_s^2}{1250} \right) s_c i_c b_c + q
\]

for a multilayer soil,

\[
q_u = (\pi + 2) \frac{(1 + \nu)}{1875B} \rho(z)V_s^2(z)dz s_c i_c b_c + q
\]

The immediate settlement is given by equation 13.

\[
s = \frac{(1 - \nu^2)B q_c f}{E_{max-m}} \quad \text{and} \quad E_{max-m} = \frac{(1 + \nu)D^{+2B} \rho(z)V_s^2(z)dz}{B}
\]

with,

\[E_{max-m}\] mean dynamic Young modulus and \(\nu\), Poisson’s ratio; \(s_c, i_c\) and \(b_c\) have the same expressions as that given above and \(c_f\) is a factor depending on the geometry of the footing.

3. APPLICATION TO THE CALCULATION OF SHALLOW FOUNDATIONS

3.1. Geotechnical and Geophysical Data

The initial geotechnical reconnaissance surveys were supplemented by investigations aimed at obtaining the dynamic geotechnical parameters, in particular the low deformations soil modules. Two cross-hole surveys were carried out, as well as 2 SPT surveys and 2 pressure meter surveys on the sites studied. These investigations were carried out on the same boreholes and at the same time under perfectly controlled conditions. The mechanical properties for characterizing the deformability of materials in "static" or very small deformations are summarized in Figure 3 for the sampling point 1 and Figure 4 for the sampling point 2. From the propagation profiles of Vs waves, it is deduced that the soils of both sites belong to soils B and C according to the classification of Eurocode 8 [3,23].

We do not claim to have obtained an exhaustive sampling that is entirely representative of all the types of soil likely to be encountered everywhere. In this study, the soil is described as a firm to stiff, becoming stiff and very stiff clay with increasing depth (see Figure 5). The results obtained on residual soils comparable to the values given in Eurocode 8 (Vs, N_SPT, c_u) [3,23] indicate some trends that already allow us to reach targeted recommendations allowing builders to use these results now "haves" comfortable security. The correlation between the \(G_{max}\) modulus and the pressure meter modulus \(E_m\) has given the two probing profiles a value of 4. A correlation between \(G_{max}/G_{static}\) (at a deformation of \(10^{-2}\)) has also been established, it has given a value of 8, which is equal at the conventional value given by many authors [20-22] and close to the conventional value of 10 given by the CFMS [24].
Figure 3 Summarized the mechanical properties for characterizing the deformability of soils in "static" or very small deformations for the sampling point 1.

Figure 4 Summarized the mechanical properties for characterizing the deformability of soils in "static" or very small deformations for the sampling point 2.
Figure 5 The soil described as a firm to stiff, becoming stiff and very stiff clay with increasing depth

3.2. Assumptions of the Calculations

The calculation of the shallow foundations is carried out on the two sites which were the object of the geotechnical and geophysical investigations described in the previous part. The loads take-down of the building provided by the design office in structural calculation at foundation level is 1320 KN. The initial geotechnical survey yielded the density at each site, which is 1833 kg/m$^3$ and a Poisson’s ratio of 0.35 was set in accordance of stiff clay [22]. Based on geotechnical and geophysical data from drill holes and foundation load value, foundation dimensions were set and adjusted based on the calculation results. The ranges of undrained shear strength $c_u$ values obtained as a function of $V_s$ are in accordance with the values given in Eurocode 8 [3,23].

In addition, the bearing parameters described in paragraphs 2.2 and 2.3; the vertical stiffness $K_z$ (for the soil - structure interaction) and the soil reaction modules $K$ were determined from the load supported and the settlement results obtained by the three methods on each of the foundations.

4. RESULTS AND DISCUSSIONS

Tables 1 and 2 present the summary of the calculation of the shallow foundations from the data of the surveys 1 and 2 and the loads take-down on each footing. These results permit us to validate the method of design of the shallow foundations from the propagation velocity of the seismic waves $V_s$. These calculations were carried out on the basis of the equations presented in part 2.2 for pressure meter and SPT methods and from the equations developed in section 2.3 for $V_s$ obtained at cross hole.
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In terms of the estimated bearing capacity, the values of \( q_u \) obtained are of the same order of magnitude for the three methods. The difference between the value obtained from the \( V_s \) method and that obtained by the SPT method is respectively -0.28\% for the foundation 1 and -6.78\% for the foundation 2. This difference is of 25.25 and 19.6\%, respectively for the foundations 1 and 2, when comparing the values obtained by the \( V_s \) method to that of the pressure meter. The values of the allowable stresses are also of the same order of magnitude. The differences between the values of \( q_u \) obtained by the SPT and pressure meter methods are respectively 25.6 and 28.3\%, respectively for the foundations 1 and 2, when comparing the values obtained by the \( V_s \) method to that of the pressure meter.

### Table 1 Calculation results of foundation by tree method for \( Q = 1320 \) kN (surveys 1: soil type at site is classified B)

<table>
<thead>
<tr>
<th>Bearing</th>
<th>SPT</th>
<th>Pressure meter</th>
<th>Cross hole</th>
<th>(*\text{CH vs } V_s (%))</th>
<th>(*\text{CH vs Pressure meter (%)})</th>
<th>Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>( q_u ) (kPa)</td>
<td>1421.95</td>
<td>1132.13</td>
<td>1417.98</td>
<td>-0.28</td>
<td>25.25</td>
<td>B = 3 m</td>
</tr>
<tr>
<td>( q_{u,ELS} ) (kPa)</td>
<td>473.98</td>
<td>391.76</td>
<td>487.04</td>
<td>2.76</td>
<td>24.32</td>
<td>L = 3 m</td>
</tr>
<tr>
<td>( q_{u,ELU} ) (kPa)</td>
<td>710.98</td>
<td>576.85</td>
<td>719.78</td>
<td>1.24</td>
<td>24.78</td>
<td>D = 1.2 m</td>
</tr>
<tr>
<td>( s ) (mm)</td>
<td>6.07</td>
<td>5.14</td>
<td>0.74</td>
<td>-87.82</td>
<td>-85.61</td>
<td></td>
</tr>
<tr>
<td>( Kz ) (MN/m)</td>
<td>217.49</td>
<td>256.91</td>
<td>1785.66</td>
<td>721.04</td>
<td>595.06</td>
<td>q_{ser} (kPa)</td>
</tr>
<tr>
<td>( K ) (MN/m³)</td>
<td>26.95</td>
<td>31.83</td>
<td>221.24</td>
<td>721.04</td>
<td>595.06</td>
<td>163.54</td>
</tr>
</tbody>
</table>

*CH vs SPT (%) or CH vs Pressure meter (%): gap in % enter the results obtained by Vs (Cross-hoie) method and STP and Pressure meter method.

### Table 2 Calculation results of foundation by tree method for \( Q = 1320 \) kN (surveys 2: soil type at site is classified C)

<table>
<thead>
<tr>
<th>Bearing</th>
<th>SPT</th>
<th>Pressure meter</th>
<th>Cross hole</th>
<th>( \text{CH vs SPT (%)} )</th>
<th>( \text{CH vs Pressure meter (%)} )</th>
<th>Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>( q_u ) (kPa)</td>
<td>886.78</td>
<td>691.18</td>
<td>826.63</td>
<td>-6.78</td>
<td>19.60</td>
<td>B = 3 m</td>
</tr>
<tr>
<td>( q_{u,ELS} ) (kPa)</td>
<td>295.59</td>
<td>263.96</td>
<td>299.52</td>
<td>1.33</td>
<td>13.47</td>
<td>L = 3 m</td>
</tr>
<tr>
<td>( q_{u,ELU} ) (kPa)</td>
<td>443.39</td>
<td>370.76</td>
<td>431.29</td>
<td>-2.73</td>
<td>16.33</td>
<td>D = 2 m</td>
</tr>
<tr>
<td>( s ) (mm)</td>
<td>5.25</td>
<td>3.51</td>
<td>1.42</td>
<td>-72.93</td>
<td>-59.58</td>
<td></td>
</tr>
<tr>
<td>( Kz ) (MN/m)</td>
<td>251.46</td>
<td>375.55</td>
<td>929.06</td>
<td>269.47</td>
<td>147.39</td>
<td>q_{ser} (kPa)</td>
</tr>
<tr>
<td>( K ) (MN/m³)</td>
<td>31.15</td>
<td>46.53</td>
<td>115.11</td>
<td>269.47</td>
<td>147.39</td>
<td>163.54</td>
</tr>
</tbody>
</table>

In terms of settlement, the values of \( s_f \) (mm) are low for the \( V_s \) method, this result is explained by the fact that the settlements obtained by the pressure meter and SPT methods were calculated in this article for a period of 50 years, whereas the settlement obtained by the \( V_s \) method is an immediate settlement (<7 days).

The significant differences between the \( V_s \) method and the other methods for vertical stiffness and reaction modulus are explained by the fact that the foundation settlement values are used to obtain vertical stiffness \( K_z \) (MN / m) = \( Q/s \) and \( K \) (MN / m³) = \( q_{ser} / s \). These values are valid in static domain. We do not recommend the calculation of \( K_z \) impedances and \( K \) reaction module from the proposed method in dynamic domain. If necessary, analytical methods already exist in the literature for calculating the impedances of a foundation in dynamic structural response [16,22,24-29]. The soil that supports a foundation structure usually has finite stiffness. It is common to apply linear spring stiffness’s, in which case the stiffness values are chosen dependent on the strain level that the soil will experience for the load case under consideration. The following shear strain levels can be expected for the three
most important sources of dynamic loading of soils: earthquakes (large strains up to \(10^{-2}\) to \(10^{-1}\)); rotating machines (small strains usually less than \(10^{-3}\)) and wind and ocean waves (moderate strains up to \(10^{-2}\), typically \(10^{-3}\)).

5. CONCLUSIONS
The method of calculating shallow foundations developed and proposed in this article has been compared to two other commonly used methods. The initial data that served as the basis for calculations were obtained on the same holes at the same time under perfectly controlled conditions. The difference between the design method from \(V_s\) that we propose and the pressure meter and SPT methods in terms of bearing capacity \(q_u\) is low. In both foundations the results obtained by the \(V_s\) method are framed by those obtained by the SPT method on one hand, and on the other hand by the pressuremeter method. The advantage of using this method lies on the fact that each type of geophysical survey involves a volume of soil whose size is related to the dimensions of the measuring device. The design method proposed from \(V_s\) is considered as an alternative method to the geotechnician. Geotechnical tests being relatively heavy and expensive on one hand, destructive on the other hand, they can only be conducted in a limited number of points. The extrapolation of geotechnical measurements to an entire geological layer and its lateral extension implicitly assumes hypotheses concerning the homogeneity of the geological layer and the distribution of mechanical properties within the layer. For reasons of cost, geotechnical engineering cannot determine and quantify the mechanical properties with a mesh of the order of \(m^2\) on a parcel of several hectares. The proposed method is reliable like all the other methods that exist for the calculation of superficial foundations. It can be used when only geophysical survey results are available on a civil engineering construction site, which are already used for calculation of foundation stiffness.

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