A refined formula for the allowable bearing pressure in soils and rocks using shear wave velocities

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Abstract

Based on a variety of case histories of site investigations, including extensive bore hole data, laboratory testing and geophysical prospecting at more than 550 construction sites, an empirical formulation is proposed for the rapid determination of allowable bearing pressure of shallow foundations in soils and rocks. The proposed expression is corroborated consistently by the results of the classical theory and its application is proven to be rapid and reliable. Plate load tests have been also carried out at three different sites, in order to further confirm the validity of the proposed method. The latter incorporates only two soil parameters, namely, the in situ measured shear wave velocity and the unit weight. The unit weight may also be determined with sufficient accuracy by means of other proposed empirical expressions, using P or S-wave velocities. It is indicated that once the shear and P-wave velocities are measured in situ by an appropriate geophysical survey, the allowable bearing pressure as well as the coefficient of subgrade reaction and many other elasticity parameters may be determined rapidly and reliably.

Keywords: shear wave velocity, shallow foundations, allowable bearing pressure, dynamic technique, soils and rocks.

1 Introduction

Professor Schulze [1], a prominent historical figure in soil mechanics and foundation engineering in Germany, stated in 1943 that “For the determination of allowable bearing pressure, the geophysical methods, utilising seismic wave velocity measuring techniques with absolutely no disturbance of natural site conditions, may yield relatively more realistic results than those of the
geotechnical methods, which are based primarily on bore hole data and laboratory testing of so called undisturbed soil samples”.

Since that time, various significant contributions have been made to the solution of geotechnical problems by means of geophysical prospecting. The P-wave velocities, for instance, have been used to determine the unconfined compressive strengths and modulus of elasticity of soil samples by Coates [2]. Hardin and Black [3], and also Hardin and Drnevich [4], based on extensive experimental data, established indispensable relations between the shear wave velocity, void ratio, and shear rigidity of soils. Similarly, Ohkubo and Terasaki [5] supplied various expressions relating the seismic wave velocities to weight density, permeability, water content, unconfined compressive strength and modulus of elasticity.

The use of geophysical methods in soil mechanics has been extensively studied for the purpose of determining the properties of soils and rocks by Imai and Yoshimura [6], Tatham [7], Willkens et al. [8], Phillips et al. [9], Keceli [10, 11], Jongmans [12], Sully and Campanella [13] and Pyrak-Nolte et al. [14]. Imai and Yoshimura [6] proposed the empirical expression

\[ nq_a = q_f = V_s^{2.4}/(1590) \] (kPa) \hspace{1cm} (1)

for the determination of the ultimate bearing capacity at failure \( q_f \) and/or the allowable bearing pressure \( q_a \), where \( V_s \) is the shear wave velocity and \( n \) a factor of safety; this expression yields values unacceptably higher than those of the classical theory as will be evident in the next section. Campanella and Stewart [15], determined various soil parameters by digital signal processing, while Butcher and Powell [16] supplied practical geophysical techniques to assess various soil parameters related to ground stiffness. An empirical expression is also proposed by Abd El-Rahman [17], for the ultimate bearing capacity of soils, using the logarithm of shear wave velocity.

A series of guidelines have also been prepared in this respect by the Technical Committee TC 16 of IRTP, ISSMGE [18] and also by Sieffert and Bay-Gress [19]. Keceli [11] and Turker [20], based on extensive case studies, supplied explicit expressions for the allowable bearing pressure using shear wave velocity. In this paper, the earlier formula presented by Tezcan et al. [21] has been calibrated and improved using the soil data from 550 construction sites. Massarsch [22] determined deformation properties of fine-grained soils from seismic tests. As regards the in situ measurement of \( P \) and \( S \)-wave velocities, various alternative techniques are available as outlined in detail by Stokoe and Woods [23], Tezcan et al. [24], Butcher et al. [25], Richart et al. [26], Kramer [27] and Santamarina et al. [28].

2 Theoretical basis for the empirical expression

In order to be able to arrive at a particular empirical expression for the allowable soil pressure \( q_a \) underneath a shallow foundation, the systematic boundary value
approach used earlier by Keceli [11] will be followed. The state of stress and the related elastic parameters of a typical soil column are shown in fig. 1. Considering a foundation depth $D_f$ with a unit cross-sectional area ($A = 1$), the typical form of the compressive ultimate bearing capacity at the base of the foundation, may be written approximately as

$$q_f = \gamma D_f$$

(2)

$$q_a = q_f/n = \gamma D_f/n$$

(3)

where $\gamma$ the unit weight of soil above the base of the foundation. In order to be able to incorporate the shear wave velocity $V_{s2}$ into the above expressions, the depth parameter $D_f$ will be expressed as velocity multiplied by time, that is,

$$D_f = V_{s2}t$$

(4)

in which, the $V_{s2}$ is intentionally selected to be the shear wave velocity measured under the foundation and $t$ is an unknown time parameter. The time parameter $t$ is introduced herein just as a dummy parameter in order to maintain consistency in the units. Substituting eqn (4) into eqn (3), yields

$$q_a = \gamma V_{s2} t/n$$

(5)

**Figure 1:** Soil column and related parameters.

| $q_a$ = Allowable stress ($q_a = q_f/n$) |
| $q_f$ = Ultimate bearing pressure $q_f$ |
| $\gamma D_f$; $q_a = \gamma D_f/n$ |

| $d$ = Settlement of layer $H$ |
| $d = P_aH/\gamma E = q_aH/E$ |
| $d = q_a/k_s$; $k_s = q_a/d$ |

Assuming $d = 0.025$ m

$k_s = q_a/0.025 = 40q_a$ (kN/m$^3$)
The unknown time parameter $t$, will be determined on the basis of a calibration process. For this purpose, a typical ‘hard’ rock formation will be assumed to exist under the foundation, with the following values for the parameters, as suggested earlier by Keceli [11],

$$q_a = 10,000 \text{ kN/m}^2, V_s^2 = 4,000 \text{ m/s}, \gamma = 35 \text{ kN/m}^3, n = 1.4 \quad (6)$$

Substituting these numerical values into eqn (5) gives $t = 0.10$ s; thus,

$$q_a = 0.1 \gamma V_s^2 / n \quad (7)$$

This is the desired empirical expression to determine the allowable bearing pressure $q_a$ in soils and rocks, once the average unit weight $\gamma$, for the soil layer above the foundation and the in situ measured $V_s^2$, the wave velocity for the soil layer just below the foundation base, are available. The unit of $V_s^2$ is m/s, the unit of $\gamma$ is kN/m$^3$, then the resulting $q_a$-value is in kPa. The unit weight values may be estimated using the empirical expressions

$$\gamma_p = \gamma_0 + 0.002 V_{p1} \quad (8a)$$

and

$$\gamma_s = 4.3 V_{s1}^{0.25} \quad (8b)$$

as proposed earlier by Tezcan et al. [21] and Keceli [29], respectively. The second expression is especially recommended for granular soils for which the measured $V_{s1}$ values represent appropriately the degree of water content and/or porosity. The wave velocities must be in m/s. The only remaining unknown parameter is the factor of safety $n$, which is assumed to be, after a series of calibration processes, as follows:

$$n = 1.4 \text{ (for } V_s^2 \geq 4,000 \text{ m/s}), \quad n = 4.0 \text{ (for } V_s^2 \leq 750 \text{ m/s}) \quad (9)$$

The calibration process is based primarily on the reference $q_a$-values determined by the conventional Terzaghi method, for all the data sets corresponding to the 550 construction sites considered. For $V_s^2$ values greater than 750 m/s and smaller than 4,000 m/s, linear interpolation is recommended. The engineering rock formations are assumed to start for $V_s^2 > 750$ m/s. The factors of safety as well as the empirical allowable bearing pressure expressions for various soil (rock) types, are given in table 1. It is seen that three distinct ranges of values are assumed for $n$. For soil types with $V_s^2 \leq 750$ m/s, the factor of safety is 4, for rocks with $V_s^2 \geq 4,000$ m/s, $n = 1.4$. For other intermediate values of shear wave velocity, linear interpolation is recommended. The validity of these values has been extensively checked and calibrated by the soil data at 550 construction sites. The relatively higher value of factor of safety assumed for soils is deemed to be
appropriate to compensate the inaccuracies and gaps existing in the measured values of shear wave velocity. In fact, Terzaghi and Peck [30] state that “the factor of safety of the foundation with respect to breaking into the ground should not be less than about 3”.

It is also found by Terzaghi and Peck [30] that the width of footing $B$ has a reducing influence on the value of allowable bearing pressure for granular soils. Therefore, a correction factor $\beta$ is introduced into the formula, for sandy soils only, as shown in the third line of table 1. The proposed values of this correction factor, for different foundation width $B$, are as follows:

\[
\beta = \begin{cases} 
1.00 & \text{for } 0 \leq B \leq 1.20 \text{ m} \\
1.13 - 0.11B & \text{for } 1.2 \leq B \leq 3.00 \text{ m} \\
0.83 - 0.01B & \text{for } 3.0 \leq B \leq 12.0 \text{ m}
\end{cases}
\]  

(10)

Table 1: Factors of safety $n$ for soils and rocks [1].

<table>
<thead>
<tr>
<th>Soil type</th>
<th>$V_s$ – range (m/s)</th>
<th>$n$</th>
<th>$q_a$ (kN/m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>‘Hard’ rocks</td>
<td>$V_s \geq 4,000$</td>
<td>1.4</td>
<td>$0.071V_s$</td>
</tr>
<tr>
<td>‘Soft’ rocks</td>
<td>$750 \leq V_s \leq 4000$</td>
<td>4.6–8.10$^{-4}V_s$</td>
<td>$0.1V_s/n$</td>
</tr>
<tr>
<td>Soils</td>
<td>$750 \geq V_s$</td>
<td>4.0</td>
<td>$0.025V_s\beta^2$</td>
</tr>
</tbody>
</table>

(1) Linear interpolation is applied for $750 \leq V_s \leq 4,000$ m/s.

(2) $\beta$, correction factor is used for sands only (eqn 10).

3 Coefficient of subgrade reaction

The shear wave velocity may be used successfully to determine $k_s$, the coefficient of subgrade reaction of the soil layer just beneath the foundation base by making use of the expressions given in fig. 1. The coefficient of subgrade reaction $k_s$, is defined, similarly to the spring constant in engineering mechanics, to be the necessary vertical pressure to produce a unit vertical displacement and expressed as,

\[
k_s = q_a/d
\]  

(11)

For shallow foundations, the total vertical displacement is restricted to 1 inch (0.025 m), as prescribed by Terzaghi and Peck [30]. When $d = 0.025$ m is substituted in eqn (11), the coefficient of subgrade reaction becomes, in kN/m$^3$,

\[
k_s = 40q_a
\]  

(12)

\[
k_s = 4\gamma V_s^2/n
\]  

(13)
4 Elasticity parameters

Once the seismic wave velocities $V_{p2}$ and $V_{s2}$ are measured by geophysical means for the soil layer No. 2, just under the foundation, several parameters of elasticity, such as the shear modulus $G$, the constraint modulus of elasticity $E_c$, the modulus of elasticity (Young’s modulus) $E$, the bulk modulus $E_k$ and the Poisson’s ratio $\mu$ may be obtained easily. The shear modulus $G$, and the constraint modulus $E_c$ are related, respectively, to the shear and $P$-wave velocities by the following expressions

$$G = \rho \, V_s^2$$

and

$$E_c = \rho \, V_p^2$$

where, $\rho$ is mass density given by $\rho = \gamma / g$. From the Theory of Elasticity, it is known that, the Young’s modulus of elasticity $E$ is related to the constraint modulus $E_c$ and also to the shear modulus $G$ by the following expressions:

$$E = E_c(1 + \mu)(1 - 2\mu)/(1 - \mu)$$

$$E = 2(1 + \mu)G$$

Using eqns (14) and (15) and also substituting $\alpha$, defined as

$$\alpha = E_c/G = (V_p/V_s)^2$$

into eqns (16) and (17) yields

$$\mu = (\alpha - 2)/2(\alpha - 1)$$

or

$$\alpha = (2\mu - 2)/(2\mu - 1)$$

The modulus of elasticity is directly obtained from eqn (17) as

$$E = (3\alpha - 4)G/(\alpha - 1)$$

The constraint modulus $E_c$ may also be obtained in terms of $\alpha$ as

$$E_c = \alpha(\alpha - 1)E/(3\alpha - 4)$$
The bulk modulus of the soil layer $E_k$ may be expressed, from the theory of elasticity, as

$$E_k = E_k = \frac{E}{3(1 - 2\mu)} \quad (24)$$

$$E_k = (\alpha - 1)\frac{E}{3} = \frac{\gamma(V_p^2 - 4V_s^2/3)}{g} \quad (25)$$

## 5 Case studies

The allowable bearing pressures have also been determined at more than 550 construction sites in and around the Kocaeli and Istanbul Provinces in Turkey, between the years 2005–2010. At each construction site, by virtue of a City by-law, an appropriate number of bore holes were drilled, standard penetration test (SPT) counts conducted, undisturbed soil samples were taken for laboratory testing purposes, where shear strength $c$, the internal angle of friction $\phi$, unconfined compression strength $q_u$ and unit weight $\gamma$ were determined. Subsequently, following the classical procedure of Terzaghi and Peck [30], the ultimate capacity and also the allowable bearing pressures were determined, by assuming a factor of safety of $n = 3$. For granular soils, immediate settlement calculations were also conducted, in order to determine whether the shear failure mechanism or the maximum settlement criterion would control the design.

The numerical values of the allowable bearing pressures $q_a$, determined in accordance with the conventional Terzaghi theory, are shown by a triangular ($\Delta$) symbol, in fig. 2, where the three digit numbers refer to the data base file numbers of specific construction sites. Parallel to these classical soil investigations, the $P$- and $S$-wave velocities have been measured in situ, right at the foundation level for the purpose of determining the allowable bearing pressures $q_a$, which are shown by means of a circle ($o$), in fig. 2. Two separate linear regression lines were also shown in fig. 2, for the purpose of indicating the average values of allowable bearing pressures determined by ‘dynamic’ and ‘conventional’ methods. In order to obtain an idea of the relative conservatism of the two methods, the ratios of allowable bearing pressures ($r = q_{ad}/q_{ac}$), as determined by the ‘dynamic’ and ‘conventional’ methods, have been plotted against the $V_s$-values in fig. 3.

It is seen that the linear regression line indicates for $V_s$-values smaller than 400 m/s a narrow band of $r = 1.03$ to $r = 1.12$, which should be regarded as quite acceptable. The ‘dynamic’ method proposed herein yields allowable bearing pressures slightly (of the order of 3 to 10 percent) greater than those of the ‘conventional’ method for $V_s$-values smaller than 400 m/s. In fact, the ‘conventional’ method fails to produce reliable and consistent results for relatively strong soils and soft rocks, because it is difficult to determine the appropriate soil parameters $c$ and $\phi$ for use in the ‘conventional’ method. Construction site numbers: 133, 134, 138, 139, 206, 207, 214, 215, 219, 502, 507 and 544, where the soil conditions have been mostly weathered andesite,
granodiorite arena, greywacke, limestone, etc. did not allow for the measurement of $c$ and $\phi$-values. Therefore, the use of ‘dynamic’ method becomes inevitable for such strong soils with $V_{s2} > 400$ m/s.

Figure 2: Comparative results of ‘conventional’ and ‘dynamic’ methods.

Figure 3: Ratios of allowable bearing pressures ($q_{a,d}/q_{a,c}$) as determined by the ‘dynamic’ and the ‘conventional’ methods.
The list of soil parameters determined by in situ and also by laboratory testing through geotechnical prospecting, as well as the in situ measured $V_p$ and $V_s$-velocities at each of the 550 construction sites, are too voluminous to be included herein. Those researchers interested in having access to these particular databases may inquire at the internet addresses: <tezokan@superonline.com>, <www.tezokan.com>.

6 Seismic wave velocities

The seismic wave velocities have been measured using $P$- and $S$-geophones by means of a 24-Channel Geometrics Abem-Pasi seismic instrument, capable of noise filtering. The $P$-waves have been generated by hitting 6 blows vertically, with a 0.15 kN hammer, onto a $250 \times 250 \times 16$ mm$^3$ size steel plate placed horizontally on ground. For the purpose of generating $S$-waves however, an open ditch of size $1.4 \times 1.4 \times 1.4$ m$^3$ was excavated and then two steel plates were placed on opposite vertical faces of this ditch parallel to the centerline of the geophones. Using the same 0.15 kN hammer, 6 heavy horizontal blows were applied onto each of these vertical steel plates. The necessary polarity of the $S$-waves was achieved by hitting these vertical steel plates horizontally in opposite directions, non-concurrently.

7 Plate load testing

For purposes of correlating the allowable bearing pressures determined by various methods, plate loading tests have been carried out at three particular construction site numbers: 335, 502 and 544. The soil parameters $c$, $q_u$, and $\gamma$ as determined by laboratory testing, as well as the $P$ and $S$-wave velocities measured at site by geophysical prospecting are all shown in table 2. A thick steel bearing plate of $316.2$ mm $\times$ $316.2$ mm $= 0.10$ m$^2$ in size is used under the test platform of size $1.50$ m by $1.50$ m. The tests are carried out right at the bottom elevations of foundations. One half of the bearing pressure $\sigma_0$, which produced a settlement of $s = 12.7$ mm was selected as the allowable pressure $q_a$ as shown in fig. 4. It is seen clearly in table 2 that the results of the proposed 'dynamic' method using $P$ and $S$-wave velocities are in very close agreement with those of the plate load testing. The allowable bearing pressures $q_a$, in accordance with the conventional theory assuming $N_c = 5.14$ and $N_q = 1$, are also calculated using

$$q_a = (cN_c + \gamma D_f N_q)/3.0 \quad (26)$$
Table 2: Comparative evaluation of allowable pressures.

<table>
<thead>
<tr>
<th>Site No Owner Lot Nos (soil type)</th>
<th>Various soil parameters ($\phi = 0$)</th>
<th>$q_a$ = allowable pressure</th>
<th>$\sigma_0$ = pressure under the test plate, kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$q_u^{(1)}$</td>
<td>$D_f$</td>
<td>$c$</td>
</tr>
<tr>
<td>335 Suleyman Turan 8 Paft./A/930 Pars. (silty clay)</td>
<td>172</td>
<td>1.5</td>
<td>86</td>
</tr>
<tr>
<td>544 Ayhan Dede G22B/574/11 (weathered diorite)</td>
<td>190</td>
<td>1.5</td>
<td>95</td>
</tr>
<tr>
<td>502 Ebru Çınar 30 L1C/440/8 (clay stone)</td>
<td>147</td>
<td>1.0</td>
<td>140</td>
</tr>
</tbody>
</table>

\(^{(1)}q_u = \) unconfined compressive strength; $n = 4$.

\(^{(2)}\)Terzaghi and Peck [30].

\(^{(3)}q_a = 0.025 \gamma_p V_s\), eqn (7).

Figure 4: Load test results at site numbers: 335, 502 and 544.
8 Numerical example

For purposes of illustration, a soft clayey soil layer of $H=15$ m beneath a shallow strip footing of depth $D_f = 2.90$ m, width $B = 1.30$ m, is considered. The in situ measured seismic wave velocities are determined to be $V_{p2} = 700$ m/s and $V_{s2} = 200$ m/s, within the soil layer just below the foundation base. By coincidence, the $P$-wave velocity within the soil layer above the foundation base is also measured to be $V_{p1} = 700$ m/s. A comprehensive set of classical soil investigations, including a number of bore hole data and laboratory testing exist for this particular site, together with the numerical values of various soil parameters ($c = 52$ kPa and $\phi = 0$), including the bearing pressure capacity determined to be $q_f = 322$ kPa by the conventional method of Terzaghi and Peck [30]. Therefore, the validity and the reliability of the proposed empirical formulae have been rigorously verified. The calculation of some elasticity parameters, using the empirical expressions presented herein, is summarised in table 3.

Table 3: Results of the numerical example; $H = 15$ m, $V_{p2} = 700$ m/s, $V_{s2} = 200$ m/s, $c = 52$ kPa, $\phi = 0$, $V_{p1} = 700$ m/s above the base.

<table>
<thead>
<tr>
<th>Formula</th>
<th>Equation</th>
<th>Numerical calculations</th>
<th>Result</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_p = \gamma_0 + 0.002V_{p1}$</td>
<td>eqn (8a)</td>
<td>$\gamma_p = 16 + 0.002 (700)$</td>
<td>17.4(1)</td>
<td>kN/m$^3$</td>
</tr>
<tr>
<td>Laboratory</td>
<td></td>
<td></td>
<td>17.2</td>
<td>kN/m$^3$</td>
</tr>
<tr>
<td>$n = 4$</td>
<td></td>
<td></td>
<td>4</td>
<td>-</td>
</tr>
<tr>
<td>$q_f = cN_c + \gamma D / N_q$</td>
<td>eqn (26)</td>
<td>$V_{s2} \leq 750$ m/s</td>
<td>318</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$q_f = 0.1V_{s2}$</td>
<td>eqn (7)</td>
<td>$q_f = 0.1(17.4)200$</td>
<td>348</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$q_0 = q_f/n$</td>
<td>eqn (3)</td>
<td>$q_f = 348 / 4$</td>
<td>87</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$k_s = 40$</td>
<td>eqn (12)</td>
<td>$k_s = 40 (87)$</td>
<td>3 480</td>
<td>kN/m$^3$</td>
</tr>
<tr>
<td>$G = \gamma V_{s2}^2 / g$</td>
<td>eqn (14)</td>
<td>$G = 17.4 (200)^2/9.81$</td>
<td>70 948</td>
<td>kN/m$^3$</td>
</tr>
<tr>
<td>$\alpha = (V_{p2} / V_{s2})^2$</td>
<td>eqn (18)</td>
<td>$\alpha = (700/200)^2$</td>
<td>12.25</td>
<td>-</td>
</tr>
<tr>
<td>$\mu = (\alpha - 2) / 2(\alpha - 1)$</td>
<td>eqn (19)</td>
<td>$\mu = (12.25-2)/(11.25)$</td>
<td>0.456</td>
<td>-</td>
</tr>
<tr>
<td>$E = 2 (1 + \mu) G$</td>
<td>eqn (17)</td>
<td>$E = 2 (1.456) 70 948$</td>
<td>206 537</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$E_c = \gamma V_{s2}^2 / g$</td>
<td>eqn (15)</td>
<td>$E_c = 17.4 (700)^2 / 9.81$</td>
<td>870 000</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$E_k = E/3 (1 - 2\mu)$</td>
<td>eqn (24)</td>
<td>$E_k = 206 537 / 3 (1 - 2\mu)$</td>
<td>774 417</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$E_s = E(\alpha - 1)/3$</td>
<td>eqn (25)</td>
<td>$E_s = 206 537 (12.25-1) / 3$</td>
<td>774 514</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>$d = \text{displacement}$</td>
<td>eqn (11)</td>
<td>$d = q_a/k_s = 87/3480$</td>
<td>0.025</td>
<td>m</td>
</tr>
</tbody>
</table>

(1) The result from eqn (8a), $\gamma = 17.4$ kN/m$^3$ is used in all subsequent expressions.
9 Discussion on the degree of accuracy

The degree of accuracy of the proposed ‘dynamic’ method is quite satisfactory and consistent as attested rigorously at more than 550 construction sites. The conventional approach however, depends heavily on the degree of accuracy of in situ and laboratory determined soil parameters. In fact, the allowable bearing pressure calculations are very sensitive to the values of $c$ and $\phi$, determined in the laboratory using so-called ‘undisturbed’ soil samples, which may not necessarily represent the true in situ conditions. This may explain the reason why at a number of construction sites, some inconsistent and erratic results for $q_a$ are obtained using the classical theory, as already depicted in fig. 2, because the laboratory measured $c$ and $\phi$-values differed considerably from one soil sample into the other. ‘Point Load’ tests of rock samples have been carried out for $V_s^2$-values greater than 400 m/s as recommended by Hunt [31].

The performed ‘Point Load’ testing is schematically shown in fig. 5 where $P_u$ is the crushing point load acting laterally, $D_e$ is the effective diameter of the soil ‘rock’ sample given by

$$D_e = (4A/\pi)^{0.5}$$

with $A$ the the cross-sectional area of the irregular soil sample and $L$ the length of soil ‘rock’ sample satisfies $L \geq 1.5D_e$. Regarding the quality parameter $r$, the values $r = 2.4$ for weathered and jointed soft rocks, $r = 7.2$ for reliable hard rock were adopted. An appropriate value is selected by engineering judgment for other samples.

![Figure 5: Point load test.](image)

For ‘hard soil’ formations, corresponding to shear wave velocities, greater than $V_s^2 > 400$ m/s, the ‘conventional’ method is unable to yield any reliable allowable soil pressure $q_a$, since neither $c$-, nor $\phi$-values may be determined in the laboratory. Any approximate approach however, using either the unconfined compressive strength $q_u$ or rock quality designation (RQD) ratios, will not be accurate enough. The ‘dynamic’ method in such cases produced consistently the same results as those obtained from the ‘Point Load’ tests. It is a fact that, the orientation of joints within a rock formation plays an important role in the in situ measured $V_s$-values. The average of $V_s$-values however, measured in various plan directions may help to improve the degree of accuracy, as recommended by Bieniawski [32]. It is true that the shear modulus, as well as the shear wave velocity of a soil layer are
reduced with increasing levels of shear strain, as reported by Massarsch [22]. The ultimate failure pressure is certainly related to very large levels of shear strains.

However, the levels of shear strains associated with allowable bearing pressure are compatible with those generated during the in situ measurement of shear wave velocities. Nevertheless, the nature of the empirical expression proposed herein for the determination of the allowable bearing pressure, using shear wave velocities measured at low shear strains, is appropriate to produce reliable results for a wide range of soil conditions. The influence of high level shear strains is considered not to be relevant to our case. Further, when the soil is saturated, the necessary to consider reduction in allowable pressure is readily expected to be taken care of by a likewise and appropriate reduction in the values of in situ measured shear wave velocities.

10 Conclusions

The P- and S-wave velocities are very significant soil properties on which a family of geotechnical soil parameters depends; these parameters range from compressive and shear strengths to void ratio and from the subgrade coefficient to cohesion, among others.

Once the shear and P-wave velocities are measured, the allowable bearing pressure, the coefficient of subgrade reaction, various other elasticity parameters as well as the approximate values of the unit weight are rapidly and economically determined, using relatively simple empirical expressions. Bore hole drilling and laboratory testing of soil samples including the ‘point load’ method of rock samples, may be beneficially used for correlation purposes.

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