Allowable Bearing Pressure in Soils and Rocks through Seismic Wave Velocities

S. S. Tezcan

Civil Engineering, Bogazici University, Bebek, Istanbul, Turkey

Tel: 90-212-352-6559 E-mail: tezokan@gmail.com

Z. Ozdemir

Bogazici University, Istanbul, Turkey

E-mail: ozdemirzuhal@yahoo.com

Received: December 20, 2011 Accepted: December 31, 2011 Published: February 1, 2012

doi:10.5539/esr.v1n1p98 URL: http://dx.doi.org/10.5539/esr.v1n1p98

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 corroborates consistently with the results of the classical theory and 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. It consists of only two soil parameters, namely, the in situ measured shear wave velocity and the unit weight. The unit weight may be also determined with sufficient accuracy, by means of other empirical expressions proposed, 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 (Schulze, W. E., 1943), 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 solving 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 (Coates, D. F., 1970). Hardin and Black (Hardin, B. O. & Black, W. L., 1968), and also Hardin and Drnevich (Hardin, B. O. & Drnevich, V. P., 1972), based on extensive experimental data, established indispensable relations between the shear wave velocity, void ratio, and shear rigidity of soils. Similarly, Ohkubo and Terasaki (Ohkubo, T. & Terasaki, A., 1976) 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 (Imai, T. & Yoshimura, M., 1976), Tatham (Tatham, R. H., 1982), Willkens, *et al*. (Willkens, R., Simmons G. & Caruso, L., 1984), Phillips, *et al*. (Phillips, D. E., Han, D. H. & Zoback, M. D., 1989), Keceli (Keceli, A. D., 1990; Keceli, A. D., 2000; Keceli, A. D., 2009), Jongmans (Jongmans, D., 1992), Sully and Campanella (Sully, J. P. & Campanella, R. G., 1995), and Pyrak-Nolte, *et al*. (Pyrak-Nolte, L. J., Roy, S. & Mullenbach, B. I., 1996). Imai and Yoshimura (Imai, T. & Yoshimura, M., 1976) proposed an empirical expression for the determination of bearing capacity q_f and / or q_a as

$$nq_a = q_f = V_s^{2.4} / (1590) \text{ (kPa)} \quad (1)$$

which yields values unacceptably much higher than the classical theory as will be evident in next section. Campanella and Stewart (Campanella, R. G. & Stewart, W. P., 1992), determined various soil parameters by digital signal processing, while Butcher and Powell (Butcher, A. P. and Powell, J. J., 1995), supplied practical geophysical techniques to assess various soil parameters related to ground stiffness. An empirical expression is also proposed by Abd El-Rahman *et al*. (Abd El-Rahman, M. M, Setto, I., & El-Werr, A., 1992), for the ultimate bearing capacity of soils, using the logarithm of shear wave velocity. A series of guidelines have been also prepared in this respect by the Technical Committee TC 16 of IRTP, ISSMGE (IRTP, 1999), and also by Sieffert (Sieffert, J. G., & Bay-Gress, Ch, 2000). Keceli (Keceli, A. D., 1990; Keceli, A. D., 2000 & Keceli, A. D., 2009), Turker (Turker, E., 2004), and Tezcan, *et al*. (Tezcan, S. S., Ozdemir, Z., & Keceli, A., 2006), based on extensive case studies, supplied explicit expressions for the allowable bearing pressure, using shear wave velocity. Massarsch (Massarsch, K. R., 2004) determined deformation properties of fine-grained soils from seismic tests. As to the in situ measurement of *P* and *S* – wave velocities, various alternate techniques are available as outlined in detail by Stokoe and Woods (Stokoe, K. H., & Woods, R. D., 1972), Tezcan, *et al*. (Tezcan, S. S., Erden, S. M., & Durgunoğlu, H. T., 1975), Butcher, *et al*. (Butcher, A. P., *et al*., 2006), Richart, *et al*. (Richart, F. E., Hall, J. R., & Woofs, R. D., 1970), Kramer (Kramer, L. K., 1996), Santamarina, *et al*. (Santamarina, J. C., Klein, A. K., & Fam, M. A., 2001), Uyanik (Uyanik, O., 2010; Uyanik, O., 2011).

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 (Keceli, A. D., 2000; Keceli, A. D., 2009) will be followed. The state of stress and the related elastic parameters of a typical soil column is shown in Figure 1. Considering a foundation depth of D_f with a unit cross-sectional area of $A=1$, the typical form of the compressive ultimate bearing capacity at the base of the foundation nothing but only as a format, may be written approximately as;

$$q_f = \gamma \; D_f \quad (2)$$

$$q_a = q_f \;/\; n = \gamma \; D_f \;/\; n \quad (3)$$

where q_f = ultimate bearing capacity at failure, γ = unit weight of soil above the base of the foundation, q_a = allowable bearing pressure, and n= factor of safety. In order to be able to incorporate the shear wave velocity $\boldsymbol{V_{s2}}$ into the above expressions, the depth parameter D_f will be expressed as velocity multiplied by time as;

$$D_f = V_{s2}\, t \quad (4)$$

in which, the V_{s2} is purposely selected to be the shear wave velocity measured under the foundation, t = is an unknown time parameter. Substituting eqn (4) into eqn (3), yields

$$q_a = \gamma\, V_{s2}\, t / n \quad (5)$$

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 parameters, as suggested earlier by Keceli (Keceli, A. D., 2000; Keceli, A. D., 2009);

$$q_a = 10\ 000 \text{ kN/m}^2,\ V_{s2} = 4\ 000 \text{ m/sec},\ \gamma = 35 \text{ kN/m}^3,\ n = 1.4 \quad (6)$$

Substituting these numerical values into eqn (5), it is obtained t = 0.10 sec, thus;

$$q_a = 0.1 \; \gamma \; V_{s2} \;/\; n \quad (7)$$

This is the desired empirical expression to determine the allowable bearing pressure q_a, in soils and rocks, once the average unit weight, γ, for the soil layer above the foundation and the in situ measured V_{s2} - wave velocity for the soil layer just below the foundation base are available. The unit of $\boldsymbol{V_{s2}}$ is in *m / sec*, the unit of γ is in *kN / m*^{3}, then the resulting q_a – value is in units of *kPa.* The unit weight values may be estimated using the empirical expressions;

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

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

$$\gamma_s = 7.6 \; (V_{s1} \; V_{p1})^{0.074} \quad (8c)$$

as proposed earlier by Tezcan *et al*. (Tezcan, S. S., Ozdemir, Z., & Keceli, A., 2006), Keceli (Keceli, A. D., 2009), and Uyanık *et al*., (Uyanik, O., Catlıoglu B., 2010) 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 units of m / sec. The only remaining unknown parameter is the factor of safety, n, which is assumed to be, after a series of calibration processes, as follows:

$$n = 1.4 \text{ (for } V_{s2} \geq 4\,000 \text{ m/sec)}, \quad n = 4.0 \text{ (for } V_{s2} \leq 750 \text{ m/sec)} \tag{9}$$

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_{s2} values greater than *750 m/sec* and smaller than *4 000 m/sec* a linear interpolation is recommended. The engineering rock formations are assumed to start for $V_{s2} > 750$ *m / sec.* 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 determined by Terzaghi and Peck (Terzaghi, K., & Peck, R. B., 1976) that the width of footing, *B,* has a reducing influence on the value of allowable bearing pressure for granular soils. Therefore, a correction factor β 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:

$$\begin{aligned} \beta &= 1.00 && \text{for } (0 \leq B \leq 1.20 \text{ m}) \\ \beta &= 1.13 - 0.11\ B && \text{for } (1.2 \leq B \leq 3.00 \text{ m}) \\ \beta &= 0.83 - 0.01\ B && \text{for } (3.0 \leq B \leq 12.0 \text{ m}) \end{aligned} \tag{10}$$

3. Coefficient of Subgrade Reaction

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

$$k_s = q_a \ / \ d \tag{11}$$

For shallow foundations, the total vertical displacement is restricted to *1 inch =0.025 m,* as prescribed by Terzaghi and Peck (Terzaghi, K., & Peck, R. B., 1976). When, *d=0.025 m* is substituted in eqn (11), the coefficient of subgrade reaction becomes in units of kN/m^3;

$$k_s = 40\ q_a \tag{12}$$

or

$$k_s = 4\gamma\ V_{s2} \ / \ n \tag{13}$$

4. Elasticity Parameters

Once, V_{p2} and V_{s2} seismic wave velocities are measured, by geophysical means, for the soil layer *No.2* just under the foundation, several parameters of elasticity, such as G = Shear modulus, E_c = Constraint modulus of elasticity, E = Modulus of elasticity *(Young'*s modulus*)*, E_k = Bulk modulus, and μ = *Poisson*'s ratio may be obtained easily. The Shear modulus, *G*, and the Constraint modulus, E_c , are related to the shear and *P*- wave velocities by the following expressions, respectively ;

$$G = \rho \boldsymbol{V_s^2} \tag{14}$$

and

$$E_c = \rho \boldsymbol{V_p^2} \tag{15}$$

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

$$E = E_c\ (1 + \mu)\ (1 - 2\mu) \ / \ (1 - \mu) \tag{16}$$

ISSN 1927-0542 E-ISSN 1927-0550

$$E = 2\ (1 + \mu)\ G \tag{17}$$

Utilising eqn (14) and (15) and also substituting α as

$$\alpha = E_c\ /\ G = (V_p\ /\ V_s)^2 \tag{18}$$

into eqn(16) and (17), we obtain

$$\mu = (\alpha - 2)\ /\ 2\ (\alpha - 1) \tag{19}$$

or

$$\alpha = (2\mu - 2)\ /\ (2\mu - 1) \tag{20}$$

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

$$E = (3\alpha - 4)\ G\ /\ (\alpha - 1) \tag{21}$$

The Constraint modulus E_c , may be also obtained in terms of α as ;

$$E_c = \alpha\ (\alpha - 1)\ E\ /\ (3\alpha - 4) \tag{22}$$

Or

$$E_c = \gamma V_p^2\ /\ g \tag{23}$$

The Bulk modulus E_k, of the soil layer, may be expressed, from the theory of elasticity, as

$$E_k = E / 3\ (1 - 2\mu) \tag{24}$$

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

5. Case Studies

The allowable bearing pressures have been also determined at more than 550 construction sites in and around the Kocaeli and Istanbul Provinces in Turkiye, between the years 2005-10. At each construction site, by virtue of City by-law, appropriate number of bore holes were drilled, SPT counts conducted, undisturbed soil samples were taken for laboratory testing purposes, where shear strength -*c*, the internal angle of friction -ϕ unconfined compression strength q_u and unit weight γ were determined. Subsequently, following the classical procedure of Terzaghi and Peck (Terzaghi, K., & Peck, R. B., 1976), the ultimate capacity and also the allowable bearing pressures were determined, by assuming the factor of safety as $n=3$. 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 (Δ) symbol, in Figure 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 Figure 2.

Two separate linear regression lines were also shown in Figure 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 about 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 Figure 3.

It is seen that the linear regression line indicates for V_s – values smaller than *400 m/sec* 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 *(on the order of 3 to 10 percent)* greater than those of the *'conventional'* method for V_s – values smaller than *400 m / sec.* 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 ϕ for use in the *'conventional'* method. At construction site *Nos: 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 ϕ - values. Therefore, the use of *'dynamic'* method becomes inevitable for such strong soils with $V_{s2} > 400\ m / sec.$

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 to have access to these particular data base, may inquire from internet <*tezokan@superonline.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 x 250 x 16 mm* size steel plate placed horizontally on ground. For the purpose of generating *S* – waves however, an open ditch of size *1.4 x 1.4 x 1.4 m* 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* – wawes was achieved by hitting these vertical steel plates horizontally in opposite directions, nonconcurrently.

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 Sites Nos: 335, 502 and 544. The soil parameters c, q_u, and γ 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 3. A thick steel bearing plate of *316.2 mm* x *316.2 mm = 0.10* square meter 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 σ_0, which produced a settlement of $s = 12.7$ *mm* was selected as the allowable pressure q_a as shown in Figure 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 are also calculated using

$$q_a = (c\, N_c + \gamma\, D_f N_q) / 3.0 \tag{26}$$

where, $N_c = 5.14$, and $N_q = 1$ for $\phi = 0$.

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/sec* and $V_{s2} = 200$ *m/sec*, 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 / sec.* 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 (Terzaghi, K., and Peck, R. B., 1976). Therefore, the validity and the reliability of the proposed empirical formulae have been rigorously verified. Calculation of some elasticity parameters, using the empirical expressions presented herein, are summarized in Table 3.

9. Discussion on the Degrees of Accuracy

The degrees of accuracy of the proposed *'dynamic'* method are quite satisfactory and consistent as attested rigorously at more than 550 construction sites. The conventional approach however, depends heavily on the degrees 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 ϕ , 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 Figure 2, because the laboratory measured c, and ϕ - values differed considerably from one soil sample into the other. The *'Point Load'* tests (Note 1) of rock samples have been carried out for V_{s2} - values greater than *400 m/sec* as recommended by Hunt (Hunt, R. E., 1984).

For *'hard soil'* formations, corresponding to shear wave velocities, greater than $V_{s2} > 400$ *m/sec* , the *'conventional'* method is unable to yield any reliable q_a – allowable soil pressure, since neither c , nor ϕ - values may not be determined in the laboratory. Any approximate approach however, using either, q_u = unconfined compressive strength or, *RQD* ratios etc, 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 (Bieniawski, Z. T, 1979). 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 (Massarsch, K. R., 2004). 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 for our case. Further, when the soil is saturated, the reduction necessary to consider 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 most powerfull soil parameters representing a family of geotechnical soil parameters, ranging from compressive and shear strengths to void ratio, from subgrade coefficient to cohesion etc,
- 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 utilised for correlation purposes.

Acknowledgements

The writers gratefully acknowledge the supply of voluminous site data by *Mr. M. Cevher,* Head, the Earthquake Research Department, Greater Municipality of Kocaeli Province, and also the scientific assistance by *Dr. Osman Uyanık,* Assoc. Professor of Geophysics at Suleyman Demirel University, Turkey.

References

Abd El-Rahman, M. M, Setto, I., & El-Werr, A. (1992). Inferring mechanical properties of the foundation material, from geophysical measurements. *Egyptian Geophysical Society*, Proceedings of the 9th Annual Meeting, 206-228.

Bieniawski, Z. T. (1979). *Engineering Rock Mass Classification,* Wiley – Interscience Publication. New York: John Wiley & Sons.

Butcher, A. P. & Powell, J. J. (1995). Practical considerations for field geophysical techniques to used assess ground stiffness. *Proc. Int. Conf. on Advances in Site Investigation Practice,* ICE London, March 1995. Thomas Telford, 701-714.

Butcher, A. P., Campanella, R. G., Kaynia, A. M., & Massarsch, K. R. (2006). Seismic cone downhole procedure to measure shear wave velocity - A guideline prepared by ISSMGE TC10: Geophysical Testing in Geotechnical Engineering. *Proc. of the XVIth International Conference on Soil Mechanics and Geotechnical Engineering*, 5p. Osaka, Japan.

Campanella, R. G. & Stewart, W. P. (1992). Seismic cone analysis using digital signal processing for dynamic site characterization. *Canadian Geotechnical Journal*, 29(3), 477-486. http://dx.doi.org/10.1139/t92-052

Coates, D. F. (1970). *Rock Mechanics Principles*. Mines Branch Monographs, No. 874.

Hardin, B. O., & Black, W. L. (1968). Vibration modulus of normally consolidated clays. *Journal of the Soil Mechanics and Foundation Division, ASCE*, 94(2), 353-369.

Hardin, B. O., & Drnevich, V. P. (1972). Shear modulus and damping in soils. *Journal of the Soil Mechanics and Foundation Division, ASCE*, 98(7), 667-692.

Hunt, R. E. (1984). *Geotechnical Engineering Investigation Manual.* New York: Mc Graw Hill Book Co.

Imai, T., & Yoshimura, M. (1976). The relation of mechanical properties of soils to P and S- Wave velocities for soil ground in Japan. Urana Research Institute, OYO Corporation.

IRTP. (1999). ISSMGE Technical Committee TC16 Ground Property Characterization from In-situ Testing, International Reference Test Procedure (IRTP) for the Cone Penetration Test (CPT) and the Cone Penetration Test with pore pressure (CPTU). *Proc. XIIth ECSMGE* Amsterdam. Balkema, 2195-2222.

Jongmans, D. (1992). The application of seismic methods for dynamic characterization of soils. *Bulletin of International Association of Engineering Geology*, 46, 63-69. http://dx.doi.org/10.1007/BF02595035

Keceli, A. D. (1990). Determination of bearing capacity of soils by means of seismic methods (in Turkish). *Geophysical Journal*, Ankara, Turkiye, 4, 83-92.

Keceli, A. D. (2000). Bearing capacity determination by seismic methods (in Turkish). *Geophysical Journal*, Ankara, Turkiye, 14(1-2).

Keceli, A. D. (2009). UCTEA Chamber of Geophysical Engineers (in Turkish). *Applied Geophysics,* Ankara, Turkiye [Online] Available at: http://jeofizik.org.tr

Kramer , L. K. (1996). *Geotechnical Earthquake Engineering*. Prentice Hall international Series in Civil Engineering Mechanics, Upper Saddle River, N. J., USA.

Massarsch, K. R. (2004). Deformation properties of fine-grained soils from seismic tests. Keynote lecture, International Conference on Site Characterization, ISC'2, 19-22 Sept. 2004, Porto, 133-146.

Ohkubo, T., & Terasaki, A. (1976). *Physical property and seismic wave velocity of Rocks*. OYO Corporation, Japan.

Phillips, D. E., Han, D. H. & Zoback, M. D. (1989). Empirical relationships among seismic velocity, effective pressure, porosity, and clay content in sandstone. *Geophysics*, 54(1), 82-89. http://dx.doi.org/10.1190/1.1442580

Pyrak-Nolte, L. J., Roy, S. & Mullenbach, B., I. (1996). Interface waves propagated along a fracture. *Journal of Applied Geophysics*, 35, 79-87. http://dx.doi.org/10.1016/0926-9851(96)00009-2

Richart, F. E., Hall, J. R., & Woofs, R. D. (1970). *Vibrations of Soils and Foundations*. Prentice Hall, Inc, Englewood Cliffs, N.J.

Santamarina, J. C., Klein, A. K., & Fam, M. A. (2001). *Soils and Waves.* New York: John Wiley & Sons.

Schulze, W. E. (1943). "*Grundbau*", Deutsche Forschungsgesellschaft für Bodenmechanik, 7th ed., B.G. Taubner Publishers, Leipzig, Germany. Also available at: Technical University of Istanbul, Issue 48, No DK624-15, Ucler Printing House, Istanbul, Turkey.

Sieffert, J. G., & Bay-Gress, Ch. (2000). Comparison of the European bearing capacity calculation methods for shallow foundations*.* *Geotechnical Engineering*, Institution of Civil Engineers, 143, 65-74. http://dx.doi.org/10.1680/geng.2000.143.2.65

Stokoe, K. H., & Woods, R. D. (1972). Insitu shear wave velocity by cross-hole method. *Journal of the Soil Mechanics and Foundation Divison*, ASCE, 98(SM5), 443-460.

Sully, J. P. & Campanella, R. G. (1995). Evaluation of in situ anisotropy from crosshole and downhole shear wave velocities measurements. *Geotechnique*, 45(2), 267-282. http://dx.doi.org/10.1680/geot.1995.45.2.267

Tatham, R. H. (1982). V_p / V_s and lithology. *Geophysics*, 47, 336-344. http://dx.doi.org/10.1190/1.1441339

Terzaghi, K., & Peck, R. B. (1976). *Soil Mechanics in Engineering Practice* (2nd ed). London: John Wiley & Sons.

Tezcan, S. S., Erden, S. M., & Durgunoğlu, H. T. (1975). Insitu measurement of shear wave velocity at Boğaziçi University Campus. *Proc. of the International Conference on Soil Mechanics and Foundation Engineering*, 2, 157-164. Istanbul Technical University, Istanbul, Turkey.

Tezcan, S. S., Ozdemir, Z., & Keceli, A. (2006). Allowable bearing capacity of shallow foundations based on shear wave velocity. *Journal of Geotechnical and Geological Engineering*, 24, 203-218. http://dx.doi.org/10.1007/s10706-004-1748-4

Turker, E. (2004). *Computation of ground bearing capacity from shear wave velocity, Continuum Models and Discrete Systems*. Eds. D. Bergman, *et al*., Netherlands, 173-180.

Uyanik, O. (2010). Compressional and shear – wave velocity measuremets in unconsolidated the top-soil and comparison of the results. *International Journal of the Physical Sciences*, 5(7), 1034-1039.

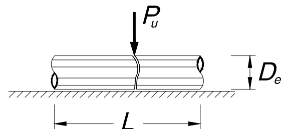
Uyanik, O. (2011). The Porosity of Saturated Shallow Sediments from Seismic Compressional and Shear – Wave Velocities. *Journal of Applied Geophysics*, 73(1), 16-24. http://dx.doi.org/10.1016/j.jappgeo.2010.11.001

Uyanik, O., & Catlıoglu B. (2010). Determination of Density from Seismic Velocities. *Proc. of the 19th International Geophysical Congress and Exhibition of Turkey*, 23-26 November 2010, Ankara, Turkey.

Willkens, R., Simmons G. & Caruso, L. (1984). The Ration V_p / V_s as a discriminant of composition for siliceous limestones. *Geophysics*, 49(11), 1850-1860. http://dx.doi.org/10.1190/1.1441598

Notes

Note 1: Point Load Testing:



q_a = Allowable bearing pressure, kN / m^2,

P_u = Crushing Point load acting laterally,

D_e = Effective diameter of the soil *'rock'* sample, $D_e = (4A / \pi)^{0.5}$

A = Cross – sectional area of the irregular soil sample,

$$q_a = r\, P_u \;/\, D_e^2$$

r = Quality parameter, $r = 2.4$ for weathered and jointed soft rocks, $r = 7.2$ for reliable hard rock. An appropriate value is selected by engineering judgement for other samples.

$L \geq 1.5\, D_e$, (L = Length of soil 'rock' sample)

Table 1. Factors of safety, n, for soils and rocks(1)

Soil type	V_s **– range** *(m/sec)*	***n***	q_a *(kN/m²)*
'Hard' rocks	$V_s \geq 4\,000$	$n = 1.4$	$q_a = 0.071\, \gamma V_s$
'Soft' rocks	$750 \leq V_s \leq 4\,000$	$n = 4.6–8.10^{-4}\, V_s$	$q_a = 0.1\, \gamma V_s / n$
Soils	$750 \geq V_s$	$n = 4.0$	$q_a = 0.025\, \gamma V_s$

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

β, correction factor is used for sands only *(eqn 10).*

Table 2. Comparative evaluation of allowable pressures

Site No Owner Lot Nos *(soil type)*	**Various soil parameters ($\phi = 0$)**						q_a **= allowable pressure**		
	q_u(1)	D_f	c	γ_{lab}	V_{p2}	V_{s2}	Terzaghi(2) *Eq. 26*	Tezcan, et.al.(3) *Eq. 7*	Load test *Fig. 4*
	kPa	*m*	*kPa*	*kN/m³*	*m/sec*	*m/sec*	*kPa*	*kPa*	*kPa*
335 **Suleyman Turan** 8 Paft./A/930 Pars. *(silty clay)*	172	1.50	86	18.9 $\gamma_0 = 16$	896	390	157	**173**	**180**
544 **Ayhan Dede** G22B / 574 / 11 *(weathered diorite)*	190	1.50	95	18.0 $\gamma_0 = 16$	1 020	453	172	**204**	**208**
502 **Ebru Çınar** 30 L1C / 440 / 8 *(clay stone)*	147	1.00	140	22.7 $\gamma_0 = 20$	1 210	489	248	**274**	**280**

(1) q_u *= unconfined compressive strength;*

(2) *Terzaghi and Peck (1976);*

(3) $q_a = 0.025\, \gamma_p V_s$ *(Eq.7), n = 4*

Table 3. Results of numerical example (H=15 m, V_{p2} = 700 m/sec, V_{s2}=200 m/sec, c = 52 kPa, ϕ =0), (V_{p1} = 700m / sec above the base)

Formula	**Equation**	**Numerical calculations**	**Result**	**Unit**
$\gamma_p = \gamma_0 + 0.002 V_{p1}$	eqn (8a)	$\gamma_p = 16 + 0.002\ (700)$	17.4[1]	kN/m^3
Laboratory	-	-	17.2	kN/m^3
n = 4	Table 1	$V_{s2} \leq 750$ m/sec	4	-
$q_f = c\ N_c + \gamma\ D_f N_q$	eqn (26)	$q_f = 52\ (5.14) + 17.2\ (2.9)\ 1$	318	kN/m^2
$q_f = 0.1\ \gamma\ V_{s2}$	eqn (7)	$q_f = 0.1\ (17.4)\ 200$	348	kN/m^2
$q_a = q_f / n$	eqn (3)	$q_a = 348 / 4$	87	kN/m^2
$k_s = 40\ q_a = 4\gamma\ V_{s2} / n$	eqn (12)	$k_s = 40\ (87)$	3 480	kN/m^3
$G = \gamma\ V_s^2 / g$	eqn (14)	$G = 17.4\ (200)^2 / 9.81$	70 948	kN/m^2
$\alpha = (V_{p2} / V_{s2})^2$	eqn (18)	$\alpha = (700 / 200)^2$	12.25	-
$\mu = (\alpha - 2) / 2(\alpha - 1)$	eqn (19)	$\mu = (12.25 - 2) / 2(11.25)$	0.456	-
$E = 2\ (1+\mu)\ G$	eqn (17)	$E = 2\ (1.456)\ 70\ 948$	206 537	kN/m^2
$E_c = \gamma\ V_{P2}^2 / g$	eqn (15)	$17.4\ (700)^2 / 9.81$	870 000	kN/m^2
$E_k = E / 3\ (1-2\mu)$	eqn (24)	$206\ 537 / 3\ (1-2\mu)$	774 417	kN/m^2
$E_k = E\ (\alpha-1)/3$	eqn (25)	$206\ 537\ (12.25-1) / 3$	774 514	kN/m^2
d = displacement	eqn (11)	$d = q_a / k_s = 87 / 3480$	0.025	m

(1) Result of eqn (8a), $\gamma = 17.4\ kN/m^3$ is used in all subsequent expressions.

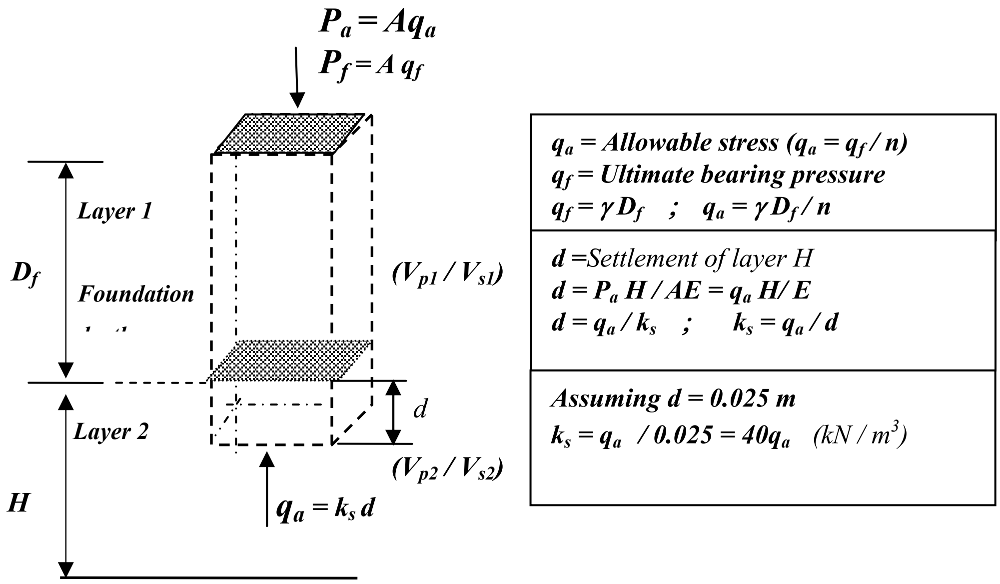


Figure 1. Soil column and related parameters

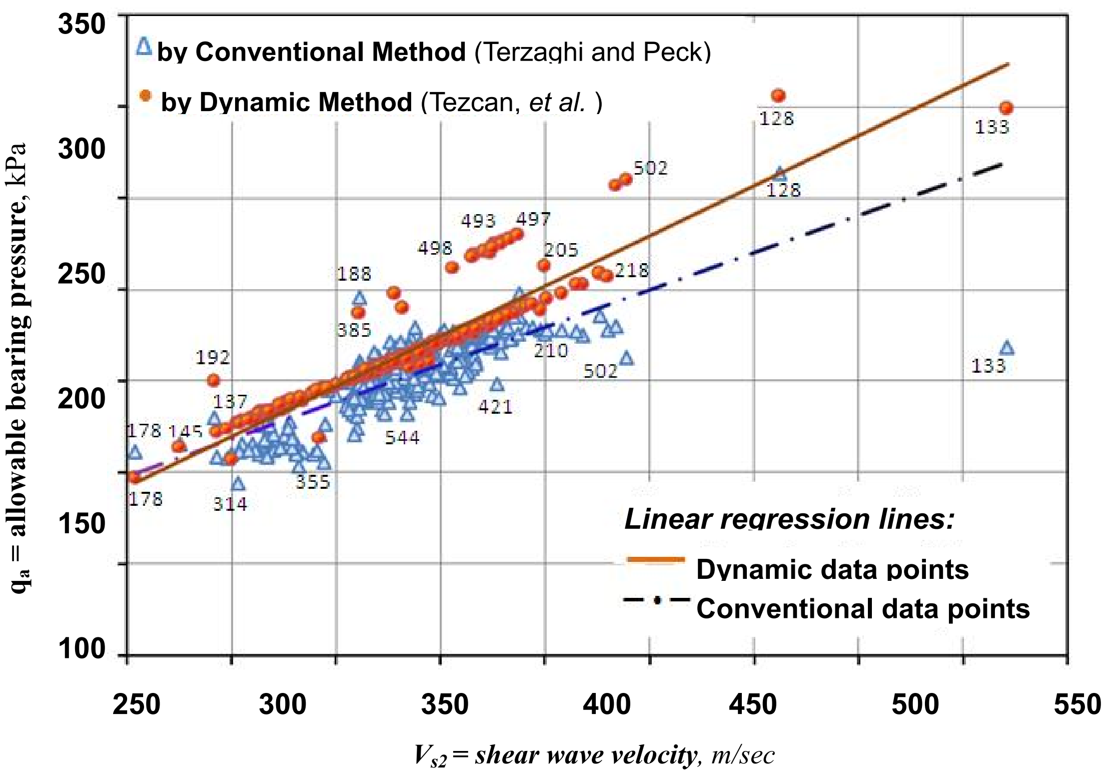


Figure 2. Comparative results of 'Conventional' and 'Dynamic' methods

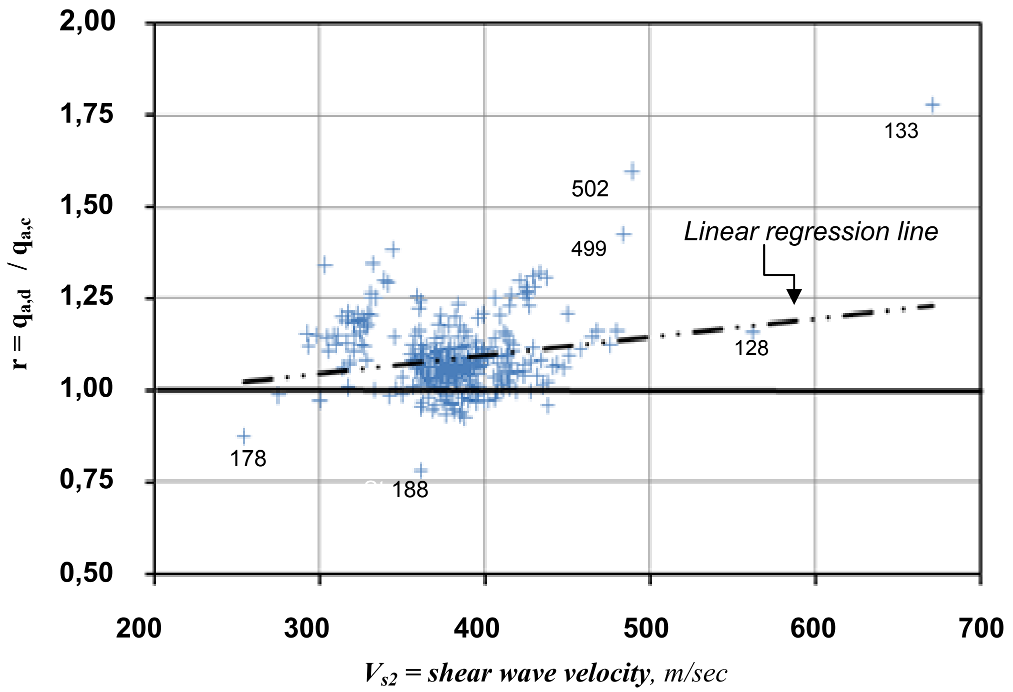


Figure 3. Ratios of allowable bearing pressures ($q_{a,d}$ / $q_{a,c}$) as determined by the 'dynamic' and the 'conventional' methods.

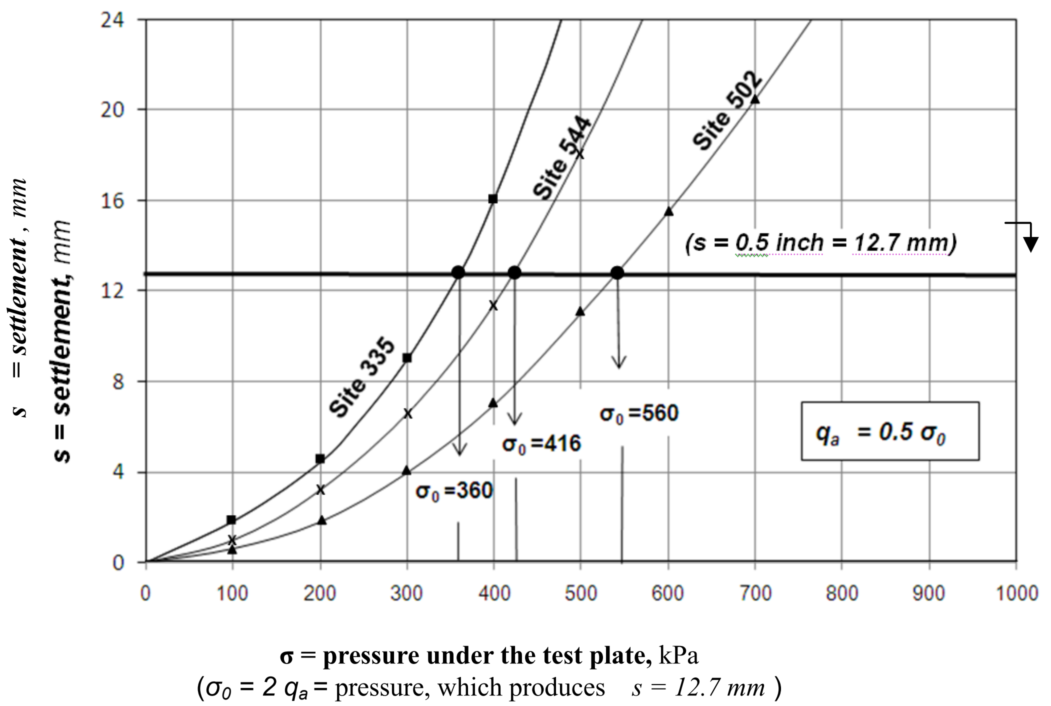


Figure 4. Load test results at Sites No: 335, 502, and 544