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Bearing Capacity Equations for Intact Argillaceous and Arenaceous Rocks in Malaysia Derived from P-Wave Velocity

Abdullah Huda Abdullah[1,*], Ismacahyadi Bagus Mohamed Jais[2]

[1] Civil Engineering Department, School of Engineering and Computing, MILA University, 71800 Nilai, Negeri Sembilan, Malaysia
[2] School of Civil Engineering, College of Engineering, Universiti Teknologi MARA, 40450 Shah Alam, Selangor, Malaysia

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ABSTRACT

This paper presents the derivation of the bearing capacity equation of shallow foundations using P-wave velocity based on the seismic refraction method on argillaceous rocks (shale) and arenaceous rocks (sandstone). Various theories of bearing capacity equations were developed over the years for the calculation of bearing capacity under vertical central loading. But, the most widely used on rock is the Mohr-Coulomb failure criterion given by Meyerhof (1963). Based on this theoretical development, the empirical equation of ultimate bearing capacity was obtained. The detailed derivation of these parameters is comprised of the results from Uniaxial Compressive Strength (UCS) and P-wave velocity values. The proposed empirical equation of bearing capacity derived from P-wave velocity herewith can be used as an alternative method in designing shallow foundations. There is a good agreement on the bearing capacity determination from P-wave velocities.

1. Introduction

In foundation engineering, bearing capacity plays an important role in determining the stability of the ground in supporting structural loads that are transmitted via a footing [1]. The stress distribution underneath the foundation provides a useful guide when deciding the extent to which an exploration should be carried out. The effectiveness of the foundation design for these structures hinges primarily on how accurately the bearing capacity of the underlying rock can be estimated [2]. Patwardhan *et al.,* [3] mentioned that the calculation of bearing capacity plays a crucial role in the design of foundations for any structure. Therefore, it is considered one of the most important performance aspects of shallow foundations on rock. The safe and economical design of the foundation is based on the concept of bearing capacity which is the ability of a rock to hold up a foundation and structure [4]. The bearing capacity of a rock is the capacity of a rock to bear the load applied by the structures constructed over it. The prediction of bearing capacity involves considering the mechanical characteristics of the rock mass, encompassing the strength and deformability of

* *Corresponding author.*
E-mail address: nurulhuda2801@gmail.com

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both the rock mass itself and the intact rock within it [5]. Moreover, the ultimate bearing capacity of a rock layer (the maximum load capacity of the rock) is the maximum load required to cause fractures in it or break it [6].

The traditional or conventional method is to use the bearing capacity equation of Terzaghi using the results from laboratory experiments, vane shear, cone penetration, pressuremeter values and Standard Penetration Test (SPT). However, one of the challenges is determining the rock parameters because these methods may be problem-related to disturbances that occurred during the sampling process, transportation and laboratory testing of the rock samples. All these procedures are also time-consuming. According to Soupios *et al.,* [7] and Adewoyin *et al.,* [8], there is an increasing requirement for geophysical surveys conducted during geotechnical investigations. These geophysical surveys provide direct information on rock quality and other geotechnical parameters that are useful in correlating geophysical results with actual rock properties. There is currently no direct relationship between ultimate bearing capacity and P-wave velocities in the prior construction area for sedimentary rock. Should an alternative method be proposed, it will have to be proven that it is to develop the bearing capacity equation using P-wave velocity based on the seismic refraction method. Therefore, the relationship between Uniaxial Compressive Strength (UCS) and P-wave velocity is to establish the sequential formulation from the strength of the material to the P-wave velocity variations into the bearing capacity equation for the shallow foundation on the rock.

2. Argillaceous Rocks (Shale) and Arenaceous Rocks (Sandstone)

Malaysia in a tropical climate region that experienced hot and humidity all the years are experiencing a high rate of the weathering process. The surficial lithology in Malaysia comprises igneous, sedimentary, and metamorphic rocks. Twenty-five percent of sedimentary rocks of Peninsular Malaysia are made up of Paleozoic black argillaceous [9]. According to the literature, rock properties are essential to be gathered in order to classify the rock material for designing projects and planning the construction procedures which will obviously sum up to the total construction cost of the rock engineering structure.

Sediments form a relatively thin surface layer of the Earth’s crust, covering the igneous or metamorphic rocks that underlie them [10]. Sedimentary rocks are rocks formed by deposition (usually underwater) of products largely formed by the destruction of pre-existing igneous rocks. They tend to be weaker than igneous rock because of the hydration of feldspars to form kaolinite and the introduction of organic minerals such as calcite. Mineralogy is the primary factor controlling the physical and chemical properties of rock [11].

Sedimentary rocks are generally classified into three groups based on the size and shape of grains of clastic rocks. Those three groups are rudaceous, arenaceous and argillaceous rocks. The constituent particles of arenaceous rocks are granules and sand meanwhile for argillaceous rocks are silt and clay. Arenaceous are the most commonly developed group among all the sedimentary rocks. They consist of grains varying between sizes of 2 and 1/16 mm and are called arenites. These rocks are commonly called sandstones. Sandstone is a sedimentary rock composed mainly of sand-size minerals or rock grains. Sandstones in which mechanically formed medium to coarse sand-sized detrital grains predominate. They form in a variety of environments and often contain significant clues regarding sorting, particle shape, and composition about their origin. Sandstones consist of a framework of detrital grains and voids. These voids may be partially or filled. Most sandstone is composed of quartz and/or feldspar because these are the most common minerals in the Earth's crust. Thus, sandstone is classified on the nature and contents of the minerals occurring in them. Sandstone mineralogy is the best indicator of sedimentary provenance: the nature of a sedimentary rock source area, its composition, relief, and location. Sandstone textures and sedimentary structures also are reliable indexes of the transportation agents and depositional setting. There are three basic components of sandstones: (1) detrital grains, mainly transported, sand-size minerals such as quartz and feldspar, (2) a detrital matrix of clay or mud, which is absent in "clean" sandstones, and (3) a cement that is chemically precipitated in crystalline form from solution and that serves to fill up original pore spaces. The colour of sandstone depends on its detrital grains and bonding material. Like sand, sandstone may have a variety of colours from grey, buff yellowish-brown, rusty brown to various shades of red. Bedding is usually obvious and sedimentary structures are common within the beds and upon the bedding surfaces. Since sandstone beds often form highly visible cliffs and other topographic features, certain colors of sandstone have been strongly identified with certain regions.

Argillaceous rocks are composed of grains sizes below or finer than 1/16 mm. They are also known as the shale. Those having grains between 1/16 and 1/256 mm are termed siltstones. Silt-size particles are mostly finely powdered clastic rocks (like rock flour), but the clay-sized particles comprise both rock flour and clay minerals (hydrous aluminium silicates). Shale is a fine-grained argillaceous sedimentary rock. It may contain various amounts of clay and silt minerals, organic matter, and precipitated salts. These fine-grained detrital rocks account for over half of all sedimentary rocks. The particles in these rocks are so small that they cannot be readily identified without appreciable magnification. In engineering applications, the content of minerals in clay includes talc, mica, chlorite or smectite. These clay minerals occur in small particle sizes and their unit cells ordinarily have a residual negative charge that is balanced by the adsorption of cations from the solution. The type of clay minerals and the availability of cations deeply affect the properties of argillaceous rocks. The deposition of shale indicates an environment of quiet and non-turbulent currents when the movement of water as the current wave is significantly reduced. As the medium is virtually fatigued or tired and cannot carry the sediment load forward anymore. Thus, leading to load dropping as a deposit. Such environments include lakes, river, floodplains, lagoons, and portions of deep-ocean basins. Even in these quiet environments, there is usually enough turbulence to keep clay-sized particles suspended for a very long time to almost indefinitely. Consequently, much of the clay is deposited only after the individual particles combine to form larger aggregates. Sometimes the chemical composition of the rock provides additional information. Black/carbonaceous shale contains abundant organic matter. When such a rock is found, it strongly implies that deposition occurred in an oxygen-poor (reducing) environment such as a swamp, where organic materials do not readily oxidise and decay. Shale exhibits the ability to split into thin layers along well-developed, closely spaced planes. This property is termed fissility. Certain shales are quarried to obtain raw material for pottery, brick, tile, and china clay. Moreover, when mixed with limestone (referred to as marl), shale is used to make Portland cement. In future, oil shale (oil trapped in tiny pore spaces) a type of shale, may become a valuable, energy resource.

Several prior researchers have conducted assessments on the physical and mechanical attributes of sedimentary rock specimens. In terms of porosity, a study conducted by Poelchau *et al.,* [12] revealed that arenaceous rock typically falls within the range of 0.25 to 0.55, while argillaceous rock exhibits a porosity range between 0.50 and 0.90. Schön [13] mentioned that the mean value of density for a selection of fourteen sedimentary rock types is 2.50 kg/m^3. Anikoh [14] reported that the average Uniaxial Compressive Strength (UCS) for argillaceous samples is 34.20 MPa. In contrast, for arenaceous samples, the mean UCS value is 57.12 MPa, with values ranging from 10 kPa to MPa. Fresh arenaceous rock, as stated by Cui and Gratcher [15], typically exhibits an average UCS of 47.5 MPa. Various researchers have noted that the lowest recorded UCS value for arenaceous rock, sourced from diverse origins and locations, is 22.8 MPa, while the highest recorded value is 157.1 MPa [16].

3. History of Bearing Capacity Equation

Bearing capacity is the most concerning factor when dealing with foundation. Rock is generally assumed to be a very good foundation due to its exceptional strength, stability, and durability. This natural material, formed through geological processes over millions of years, possesses unique properties that make it an ideal choice for various construction and engineering applications. As corroborated by previous research conducted by Gül and Ceylanoğlu [17], rock is generally assumed to be a very good foundation. However, overload leads to considerable subsidence or sudden failures in rock masses. Thus, as in the design of the foundation on the ground, much attention and care should be given to the design of the foundation to be constructed on rock masses. Over the years, several equations have been formulated to compute the bearing capacity when subjected Over the years, several equations have been formulated to compute the bearing capacity when subjected to vertical central loading [18].

3.1 Bearing Capacity on Mohr-Coulomb Failure Criterion

In 1943, Terzaghi was the first to present a comprehensive theory for the evaluation of the ultimate bearing capacity of a rough shallow foundation. Terzaghi's bearing capacity equation is broadly used in predicting the bearing capacity of soil as well as for rock, by assuming the failure behaviour follows the Mohr-Coulomb criteria. It is based on the combination of the Mohr theory of strength and the Coulomb equation. This limit equilibrium method is a superposition of the three terms; cohesion (cNc), surcharged (cN_q)and the self-weight of the ground material $\left(0.5\gamma BN_\gamma\right)$. Therefore, Terzaghi's equation can be written as,

$$q_{ult} = cN_c + \gamma DN_q + 0.5\,\gamma BN_\gamma, \qquad \text{Mohr-Coulomb Failure Criterion} \tag{1}$$

where the ultimate bearing capacity, q_{ult} , cohesion of soil/rock, c, unit weight of soil/rock, γ, depth of embedment, D, breadth of the footing, B and bearing capacity factors, N_c, N_q, N_γ respectively.

However, for strip footing resting on the ground surface, the second term does not exist where the depth, D is equal to zero. After the development of Terzaghi's derived equation, several researchers studied and modified the equation (Meyerhof (1951) and (1963), Hansen (1970), Vesic (1973) and Bowles (1996)). It shows that the bearing capacity factors did not change much and were presented in the form of tables or charts by Bowles [19]. To observe the differences between the factors, a comparison chart representing the different bearing capacity of these authors is shown in Figure 1. A plot of bearing capacity factors of the above authors, in normal scale axes, are presented.

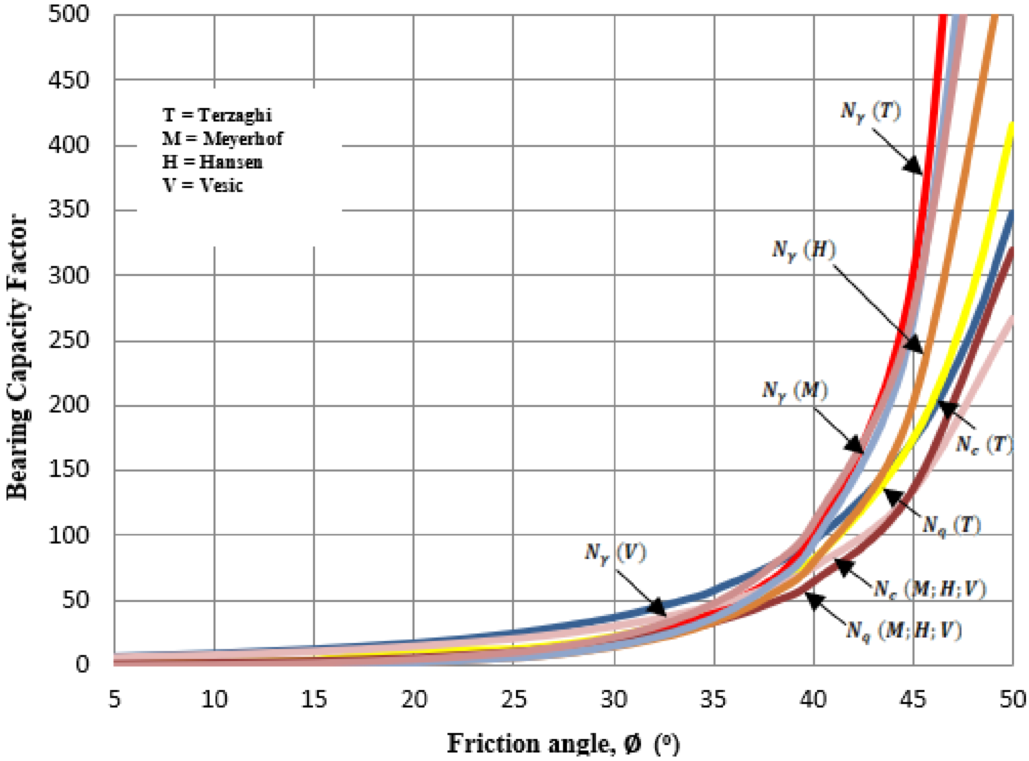

Fig. 1. Comparison of bearing capacity factors on a normal scale [19]

It was found that N_γ values have the widest suggested range of values compared to the other bearing capacity factors. The dependency of bearing capacity factors on the friction angle is very noticeable. The non-linear correlations of N_c, N_q and N_γ to the friction angle show that the bearing capacity increases with friction angle and tremendously increases when the friction angle is above 30°. For rock, the friction angle of 30° is commonly adopted in designing rock structures [13].

Alencar *et al.,* [20] mentioned that the bearing capacity of the rock mass is usually related to the condition of the structural geology. He noted that bearing capacity factors determined from laboratory experiments exhibit higher bearing capacity than the theoretical value, particularly for rough bearing surface and high friction angle. Apart from the ultimate value, the allowable value of bearing capacity has also been correlated with the intact strength as supported by Zheng *et al.,* [21] Their numerical analysis demonstrates that the fissures in rock mass are generally the main cause for the reduction of the bearing capacity of rock foundations. By introducing the upper and lower bound approach, they predicted the bearing capacity factors, N_c of two different conditions of fissured rock with their respective upper and lower boundaries, as shown in Figure 2. Additionally, the bearing capacity factor of fissured rock with vertical and horizontal fissures exhibits a lower value than the vertical fissured rock. This indicates that orientation is a critical influence on the bearing capacity of a fissured rock. It is assumed that the value of bearing capacity shall be reduced significantly if both conditions of fissured rock exist in an inclined orientation.

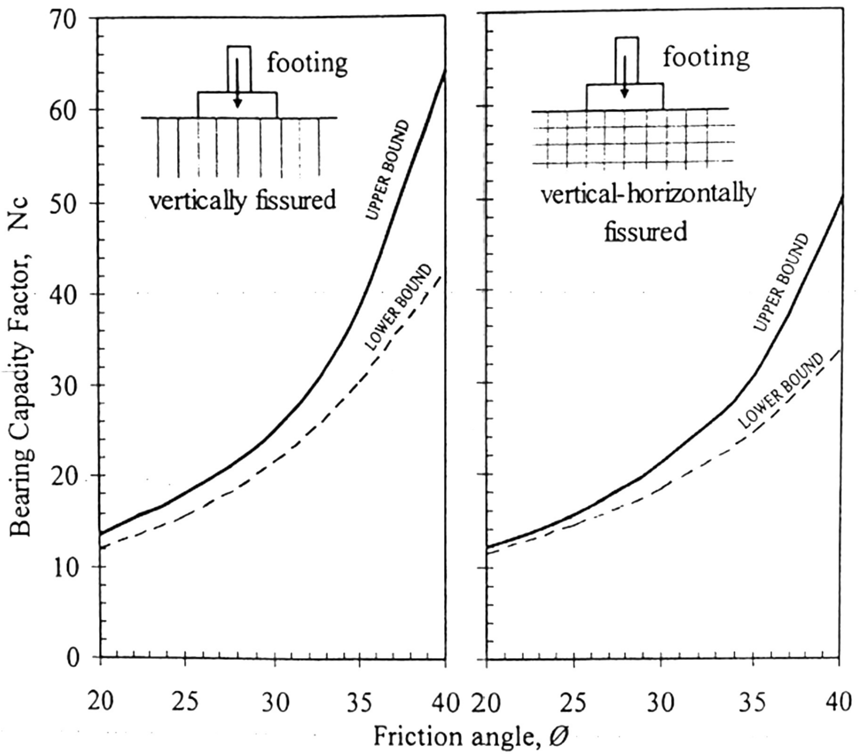

Fig. 2. Bearing capacity factors of fissured rock, N_c [19]

3.2 Bearing Capacity on Hoek-Brown Failure Criterion

The Hoek–Brown failure criterion is an empirically derived relationship used to describe a non-linear increase in peak strength of isotropic rock with increasing confining stress. According to Hoek and Brown (1988) theory, the failure criterion for some empirical methods was developed to calculate the bearing capacity of the rock foundation on jointed rock masses. Carter and Kulhawy [22] proposed a simple lower-bound solution for the bearing capacity of a weightless rock mass obeying a non-linear Hoek-Brown yield criterion. Thus, the bearing capacity of the strip footing is given as:

$$q_{ult} = [S^{0.5} + (m_b s^{0.5} + s)^{0.5}]\sigma_c \tag{2}$$

where ultimate bearing capacity,q_{ult}, rock mass strength constants, s and m_b, discontinuity spacing, S, uniaxial compressive strength of the intact rock, σ_c respectively.

The magnitude of m_b and s depends on the Geological Strength Index (GSI) which characterizes the quality of the rock mass. It can be computed empirically as mentioned in El-Naqa [5] and take the form:

$$s = \exp\left(\frac{GSI - 100}{9 - 3D}\right) \tag{3}$$

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right) \tag{4}$$

The strength parameter of intact rock, m_i depends on the rock types which are texture and mineralogy while a coefficient of disturbance, D ranges between 0 for undisturbed rock and 1 for totally disturbed.

Hoek and Brown (1997) in Hoek *et al.,* [23] have proposed values of m_i for sedimentary, metamorphic and igneous rock. In respect to sedimentary rock, the values ranged between 22 for conglomerate as coarse texture rock and 4 for claystone as very fine texture rock, as tabulated in Table 1 [22].

Table 1
Values of strength parameter, m_i for clastic sedimentary rock [24]

Class	Texture			
	Coarse	Medium	Fine	Very fine
Clastic	Conglomerate 22 Greywacke 18	Sandstone 19	Siltstone 9	Claystone 4

Serrano *et al.,* [19] used the modified Hoek-Brown criterion to predict the bearing capacity which can be expressed as:

$$q_{ult} = (N_\beta - \zeta_n) A_n \sigma_c \tag{5}$$

where bearing capacity factor, N_β, rock mass stiffness, ζ_n strength modulus constant, A_n respectively.

The bearing capacity factor, N_β can be determined graphically using $(N_\beta - \zeta_n)$ chart as shown in Figure 3 while ζ_n and A_n are derived as:

$$A_n = 0.125\, m_b \tag{6}$$

$$\zeta_n = \frac{s}{m_b A_n} \tag{7}$$

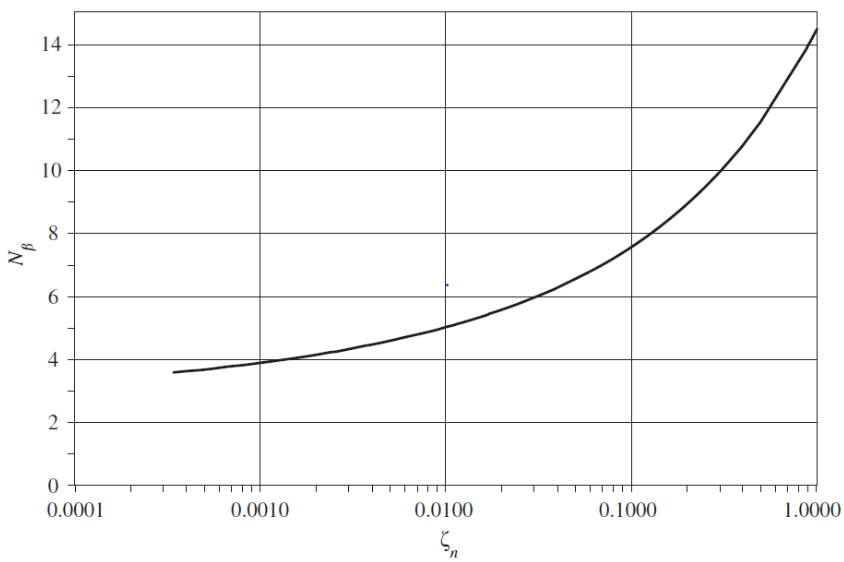

Fig. 3. Correlation of bearing capacity factor, N_β to rock mass stiffness, ζ_n [25]

The Hoek material constant of m_b and s was previously described. By considering a parameter of $(N_\beta - \zeta_n)$ as a multiplication factor, the ultimate bearing capacity of the rock mass can be determined. This correlation shows another approach to estimating the multiplication factor for the prediction of the ultimate bearing capacity rock masses.

Merifield *et al.,* [26] predicted the bearing capacity of rock mass by numerical modelling using upper and lower bound limit theorems. He predetermined the rock mass strength used in the modelling by using the Hoek-Brown failure criteria. Hence, the graphical plots were produced correlating the material constant mi to the Geological Strength Index (GSI) and bearing capacity factor, N_σ as shown in Figure 4. He recommended the graph be used in predicting the bearing capacity of rock mass for the design of shallow foundations resting at the ground surface. From the graph, it shows that as the quality of rock improves (m_i), the GSI values increase. It is expected that as the bearing capacity factor, N_σ increases, the bearing capacity of the rock also increases.

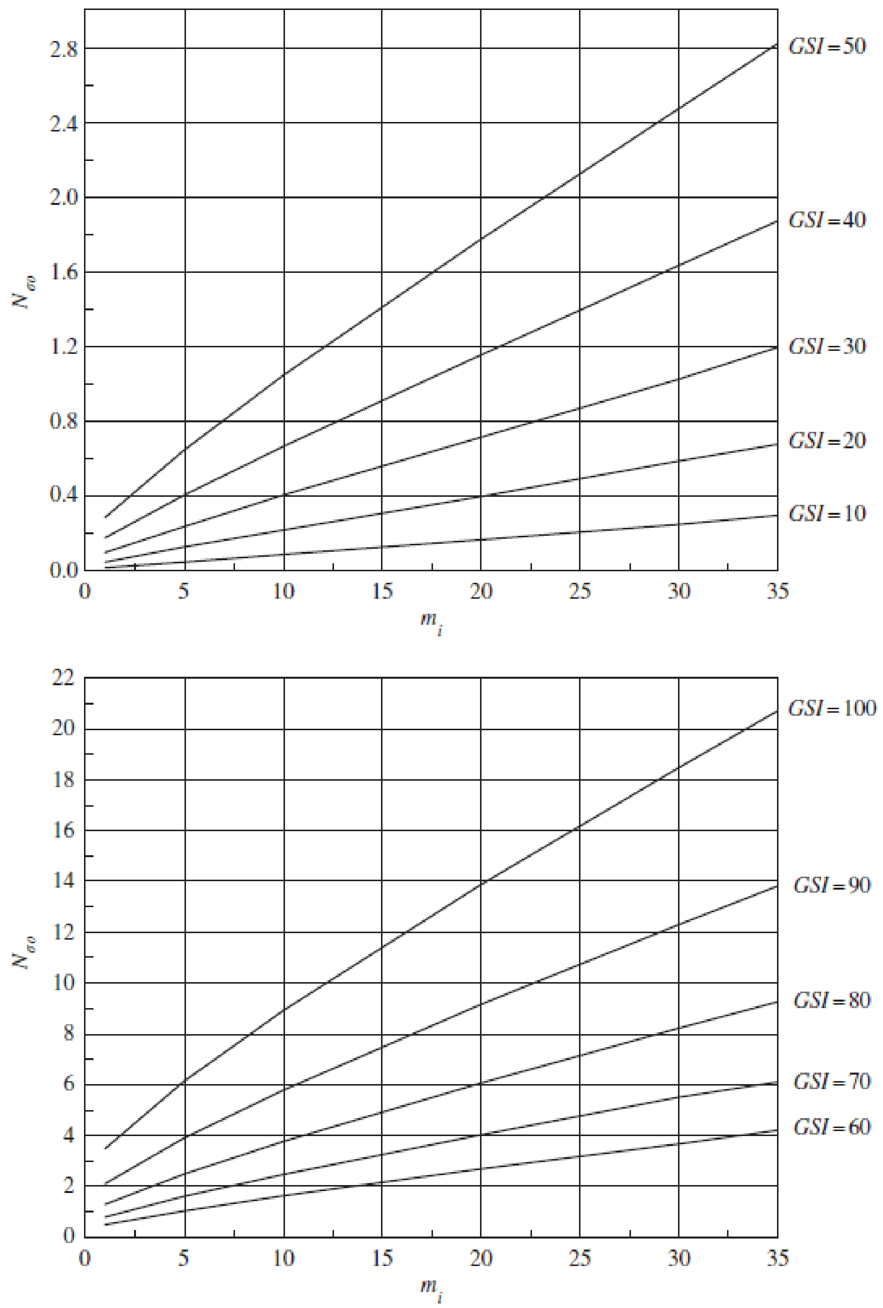

Fig. 4. Correlation of rock mass constant, m_i to GSI and N_σ [26]

3.3 Bearing Capacity on Rock Strength

The uniaxial compressive strength of intact rock has always been considered the most reliable index for strength estimation of rocks and is commonly used in empirical relationships to establish the bearing capacity of the rock mass [27]. Many attempts have been made from previous studies in Zhang [28] to correlate the ultimate bearing capacity with the compressive strength of intact rock using two assumptions. The first is by assuming that the rock mass which is influenced by confinement, exhibits higher strength as compared to the uniaxial compressive strength of intact rock determined in a laboratory. Secondly is by considering the strength of rock mass is lower than the unconfined strength of intact due to the discontinuous effects.

Zhang and Einstein [29] examine that the ultimate bearing capacity of the rock mass and found that it is linearly correlated with the uniaxial compressive strength of intact rock material by some multiplication factors and can be expressed as,

$$q_{ult} = N_\sigma \sigma_c \ , \tag{8}$$

where ultimate bearing capacity, q_{ult}, bearing capacity factor, $N_{\sigma,}$ uniaxial compressive strength of intact material, σ_c respectively.

The values of N_σ introduced by other researchers as tabulated in Table 2, vary from 1.0 to 8.0. The limitation of the prediction to the types and conditions of rock was not clearly described and it is believed that the proposed equations were not intended for weak and fractured rocks. However, Ramamurthy (1995) in Ramamurthy, [30] found that, based on the model studies of footing on rock material, the ultimate bearing capacity is taken as 1.4 times the compressive strength of intact material.

Table 2
Relationship between q_{ult} and σ_c [29]

Bearing capacity equation, $q_{ult} = N_\sigma \sigma_c$	References
$q_{ult} = 8.0\ \sigma_c$	Teng (1962)
$q_{ult} = 3.0\ \sigma_c$	Coates (1967)
$q_{ult} = 2.7\ \sigma_c$	Rowe and Armitage (1987)
$q_{ult} = 4.5\ \sigma_c \leq 10$ MPa	ARGEMA (1992)
$q_{ult} = (1 \text{ to } 4.5)\sigma_c$	Findlay et al. (1997)

Goodman [31] suggested that the ultimate bearing capacity of homogenous rock is expressed:

$$q_{ult} = (N_\emptyset + 1)\sigma_c \tag{9}$$

where ultimate bearing capacity, q_{ult}, $N_\emptyset = \ tan^2\left(45 + \frac{\emptyset}{2}\right)$, uniaxial compressive strength of intact material, σ_c respectively.

The multiplication factor, $(N_\emptyset + 1)$ is a function of the friction angle. It means that rocks with higher friction angle generate a higher value of $(N_\emptyset + 1)$, hence presents a higher ultimate bearing capacity. Attempt to compute the value of $(N_\emptyset + 1)$ concerning friction angle of 20° to 45°, gives the multiplication factor ranging between three and six. This factorised value displays similar ranges to that expressed by Zhang and Einstein, [29] in Table 2.

On the other hand, data from shaft loading test on sedimentary rocks from numerous sources provide some input on the uniaxial compressive strength of intact rocks and maximum bearing capacity of the respective weak rocks, as presented in [29]. Figure 5 shows the tabulation of the data exhibited by mudstone, shale, sandstone, clay-shale, gypsum, hardpane, till, marl, limestone and diabase compiled from several researchers. Based on these data, Zhang and Einstein, [29] explored the best-fit correlation of ultimate bearing capacity with the unconfined compressive strength of intact rock and suggested that,

$$q_{ult} = 4.83\, {\sigma_c}^{0.5} \tag{10}$$

This correlation is reliable/useable provided the ultimate bearing capacity is about 6.0 to 1.5 times of its uniaxial compressive strength.

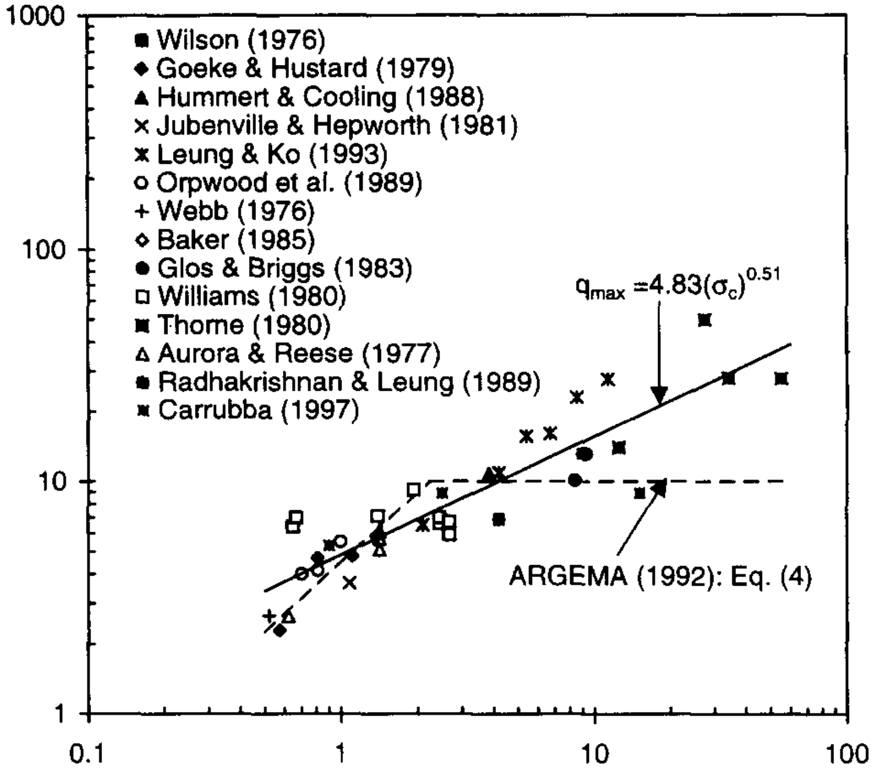

Fig. 5. Correlation of q_{ult} to σ_c for sedimentary rock as derived from [29]

Based on Irfan and Powel, [32], the modification factor is 0.2 with the assumption that the strength of rock mass is lower than the compressive strength of intact rock due to discontinuous effect. The Canadian Geotechnical Society, CGC (1985) in Zhang [28] considered the modification factors as 0.1 and 0.25 for joint spacing in the range of 0.3 m to 1.0 m and 1.0 m to 3.0 m respectively. It indicates that the rock mass with closed joint spacing generates a lower bearing capacity of rock as compared to widened joint spacing.

Meanwhile, Bowles, [19] suggested that the ultimate bearing capacity can be adopted at 33 % of the uniaxial compressive strength of intact rocks for Rock Quality Designation (RQD) less than 70 %. For rock mass having RQD of more than 70 %, the ultimate bearing capacity can be estimated between 33 % and 80 % of the ultimate bearing capacity of the intact rock material. RQD represents the quality of rock masses to discontinuities. The RQD value of less than 70 % represents fair to very poor quality of rock while RQD more than 70 % represents good to the excellent quality of rock. The rock mass with more discontinuities or lower RQD value exhibits a lower compressive strength as compared to the compressive strength of intact rock, hence, reduces the ultimate bearing capacity of the rock mass.

El-Naqa, [5] presented a set of data on computed ultimate bearing capacity and its respective uniaxial compressive strength of intact rock for jointed limestone and jointed sandstone. A combination of both data presents a better correlation representing a jointed sedimentary rock. The best-fit correlation of q_{ult} and σ_c for each type of rock is the powered correlation as given by:

$$q_{ult} = 0.22\, {\sigma_c}^{1.18} \tag{11}$$

4. Geophysics in Relation in Rock Material

Currently, geophysical methods are widely used for engineering site characterization [33]. According to Meric *et al.,* [34], geophysical properties (primarily seismic velocity) can vary with the nature of the geologic formation and degree of fracturing and weathering, as well as the presence of water in the rock mass. For example, an increase in the degree of fracturing leads to a decrease in the P-wave velocity value. The geophysical techniques which can be used in the ground to identify the velocity value of the ground's elastic disturbance was suggested by Abdullah *et al.,* [35] and is called the seismic technique.

Seismic refraction is one of the geophysical methods to measure the thickness of the weathered rock cover. Seismic waves travel with different velocities in different rocks. This method yields only zoning of depth in terms of longitudinal velocities. It has proved extremely useful as a site investigation tool for rapid comparison between several sites. By generating seismic waves and measuring the time required for the waves to travel from the geomaterial, it can determine the velocity distribution and subsequently, the nature of the subsurface layers. The travel time of seismic refracted waves follows the law of refraction (Snell's law) when they encounter a stiffer material in the subsurface. It is measured so that the wave propagation velocity and the corresponding stiffness of the geomaterial are determined. Figure 6 illustrates a schematic diagram of a seismic refraction test in a two-layered model. Additionally, the propagation path of the seismic waves can be seen.

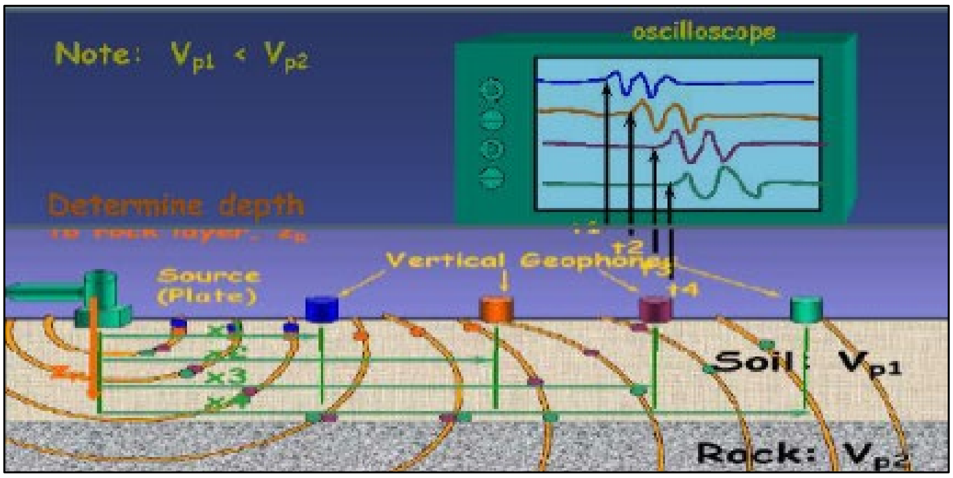

Fig. 6. The path of seismic wave's refraction from source to geophone [36]

Generally, there are three main components in the seismic refraction survey: the seismic source, seismic receiver and seismic record. The equipment was correctly set up to collect data for the seismic refraction survey. The wave has been generated using a seismic source which is a sledgehammer. Otherwise, the geophone is one device that could detect the seismic wave and receive the wave without storing the data will be useless and no analysis could be made. Hence, the device could store the data obtained from geophones in the seismogram. The geophones used can have natural frequency of 14 Hz or above.

Based on the previous study, there have many studies conducted to correlate the rock material and P-wave velocity values. The typical values of P-wave velocity for different earth materials illustrate key differences as shown in Table 3 [37].

Table 3
P-wave velocity typical values for different earth materials [37]

Material	P-wave velocity (m/s)
Air	330
Water	1450
Sands and clays	3000 - 1900
Glacial till	1500 - 2700
Chalk	1700 - 3000
Strong limestone	3000 - 6500
Weathered granite	1000 - 6000
Fresh granite	3000 - 6000
Slate	5000 - 7000

5. Result and Analysis

Data were collected from two different site areas which are in Gopeng and Putrajaya. A total of 24 rock samples with various depths were prepared for laboratory testing. Both sites possessed arenaceous and argillaceous rock samples. In this study, all rock samples were established to have UCS and P-wave velocity values. Generally, the rock samples were prepared with the length-to-diameter ratio 2:1 of the cylindrical specimen to validate UCT testing. In this study, the cylindrical specimen has a diameter of 54 mm. The stress value at failure is defined as the compressive strength of the specimen.

Subsequently, the relationship between UCS, P-wave velocity and density values were plotted as shown in Figure 7 to Figure 10. The high regression coefficient (i.e. R^2 = 0.9) reveals a strong correlation between the UCS and P-wave velocity and P-wave velocity and density which enables the estimation of one parameter having another one. According to the findings, the equation for arenaceous and argillaceous from the best straight line is illustrated in Table 4.

Table 4
Correlation between P-wave velocity, UCS and density in this study

Sedimentary rocks	P-wave Velocity – UCS relations	P-wave Velocity – Density relations
Argillaceous rock	$UCS = 0.0127V_P - 1.7208$	$\rho = 0.6297\,V_p + 673.64$
Arenaceous rock	$UCS = 0.024V_P - 32.772$	$\rho = 0.1629\,V_p + 1978.5$

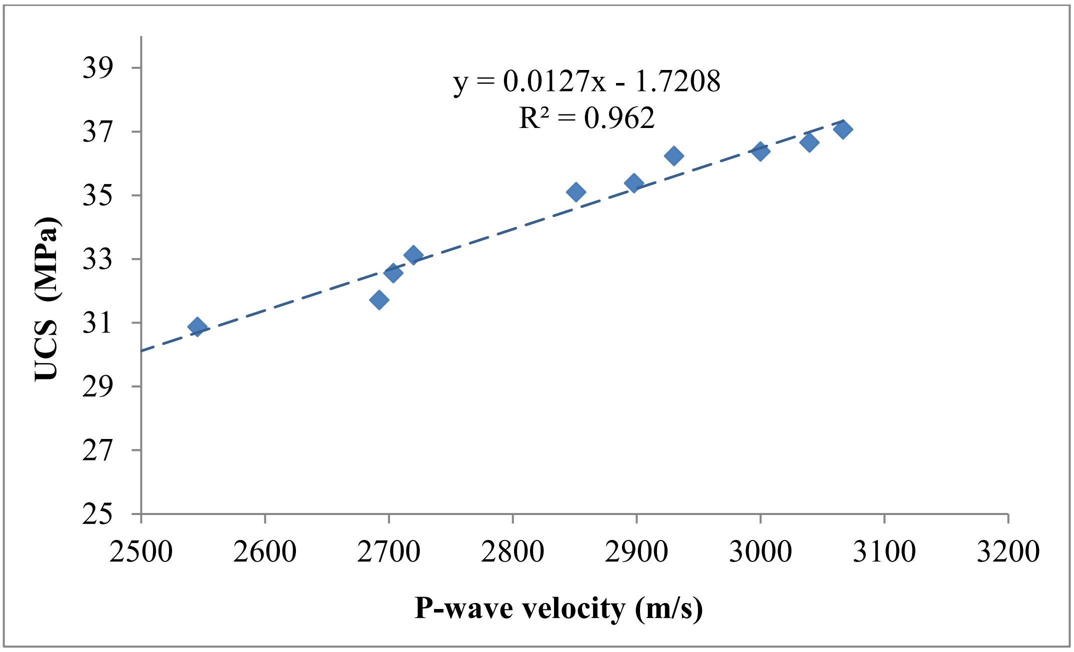

Fig. 7. Correlation between UCS and P-wave velocity (v_p) of argillaceous rock in this study

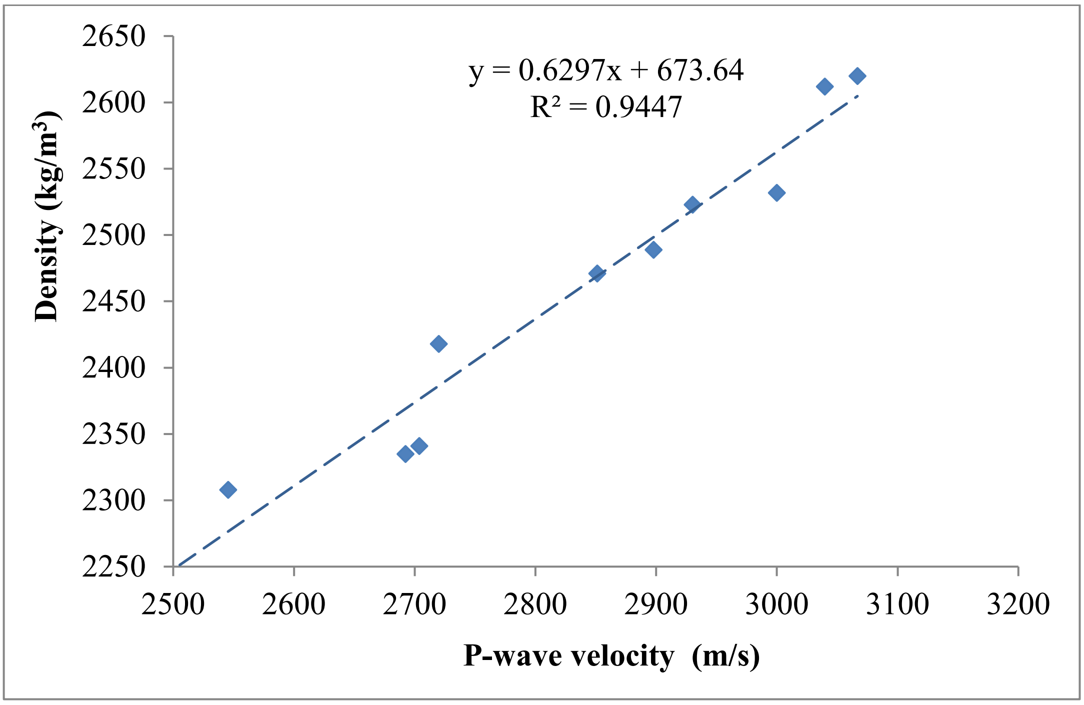

Fig. 8. Correlation between P-wave velocity (v_p) and density (ρ) of argillaceous rock in this study

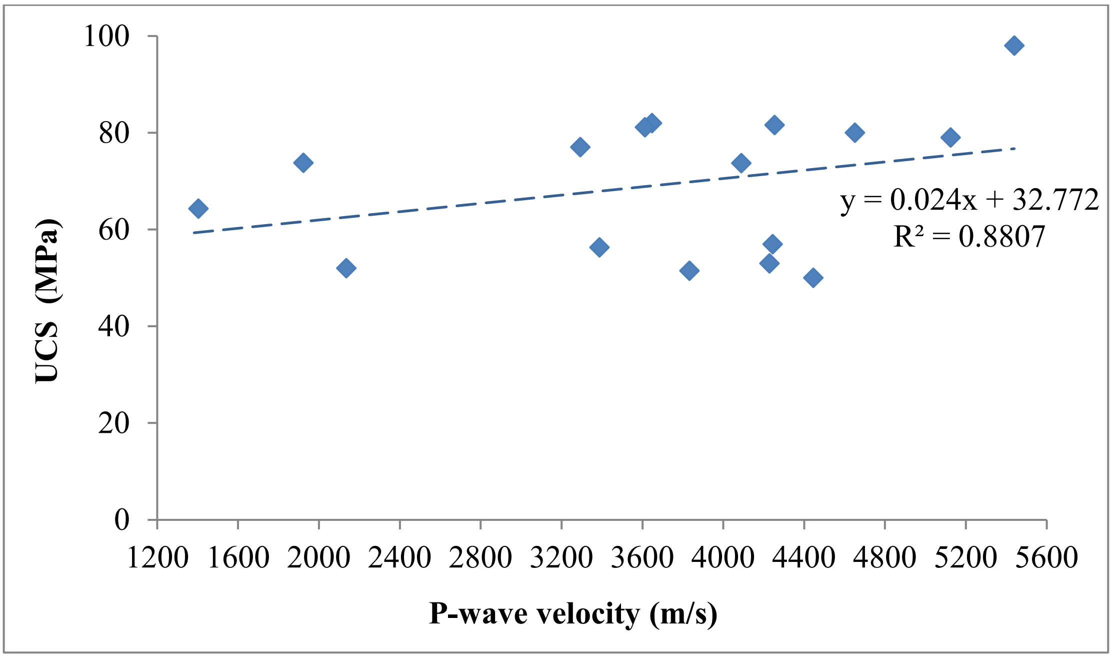

Fig. 9. Correlation between UCS and P-wave velocity (v_p) of arenaceous rock in this study

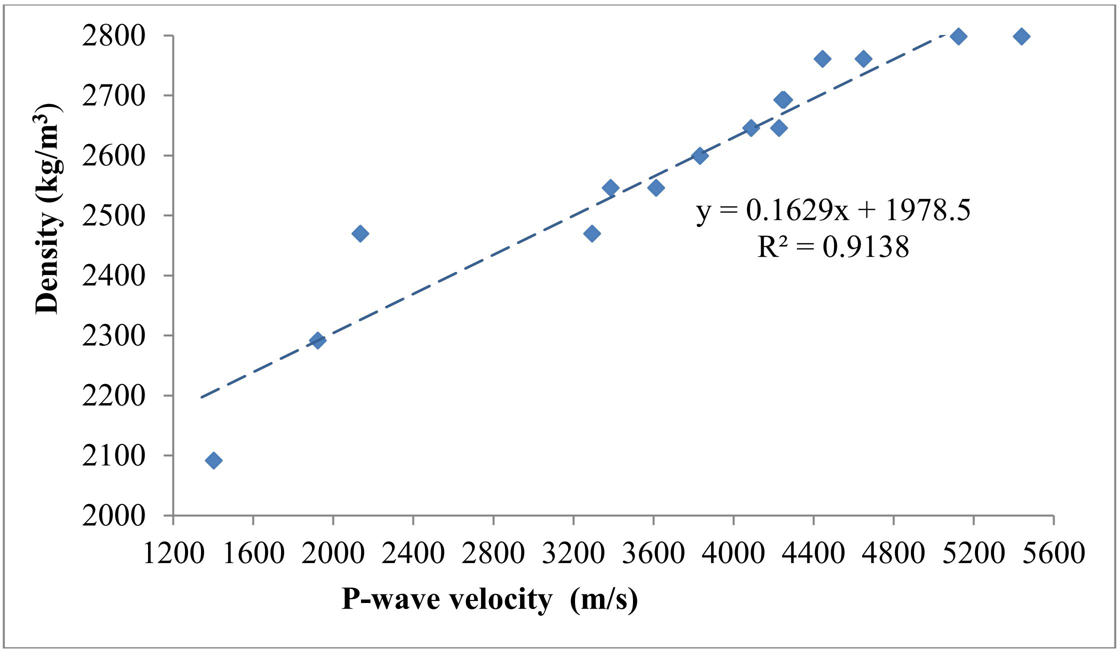

Fig. 10. Correlation between P-wave velocity (v_p) and density (ρ) of arenaceous rock in this study

5.1 Development of Bearing Capacity Equation

Several theories of bearing capacity analysis have been applied to rocks but the most widely used is the Mohr-Coulomb theory. An important term in the bearing capacity equation is the ultimate bearing capacity. The ultimate bearing capacity is defined as the maximum load required to cause fractures on a rock or break it. The equation for calculating the bearing capacity of the shallow foundations of a long strip foundation on a rock mass using the Mohr-Coulomb failure criterion is given by Meyerhof (1963). It has three terms, each comprising of bearing capacity factors and correction coefficient factors. The following equation is the general bearing capacity equation:

$$q_{ult} = c' N_c F_{cs} F_{cd} F_{ci} + \gamma D N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \tag{12}$$

In this theory, the correction coefficient factors in Eq. (1) are ignored and can be assumed equal to one. Therefore, the simplified form of the equation is:

$$q_{ult} = c' N_c + \gamma D N_q + \frac{1}{2} \gamma B N_\gamma \tag{13}$$

The equation approach is based on the three terms: cohesion, depth and overburden pressure and width and length of the foundation. Therefore, the foundation size also affects the bearing capacity of a shallow foundation on the rock materials.

For foundation design, it may be satisfactory to use the typical value of friction angle of arenaceous and argillaceous rocks is 25°as those given by after Willie, [38]. Therefore, referring to the table from Meyerhof's method, the bearing capacity factors have been determined and given as:

$$N_c = 20.71;\ N_q = 10.7;\ N_\gamma = 6.8$$

Rearranging Eq. (2) with the substitution of bearing capacity factors gives:

$$q_{ult} = 20.17\, c' + 10.7\, \gamma D + 3.4\, \gamma B \tag{14}$$

The relationship between the cohesion of the rock, c' and UCS can be written as:

$$c' = \frac{UCS}{2}$$

Meanwhile, the simplified unit weight of the rock is:

$$\gamma = \rho g$$

where unit weight of the rock, γ , density of the rock, ρ , gravity of the rock, g, assuming as 9.81 m/s^2 respectively.

By substituting these two terms of $c'$and γ in Eq. (2), the suggested equation is given in the following equation:

$$q_{ult} = 10.09\, UCS + 104.97\, \rho D + 33.35\, \rho B \tag{15}$$

All the above-mentioned parameters have some influences on ultimate bearing capacity. Therefore, P-wave velocity is expected to have a relationship with the ultimate bearing capacity. In terms of the estimated ultimate bearing capacity, the values of q_{ult} obtained are the same derivations from Meyerhof's theory.

From the results of the Uniaxial Compression Test (UCT) and Portable Ultrasonic Non-destructive Digital Indicating Tester (PUNDIT) tests, the relationship developed between Uniaxial Compressive Strength (UCS) value, P-wave velocity (V_p) and the density (ρ) of rock material is determined as:

$$UCS = 0.0127 V_P - 1.7208$$

$$\rho = 0.6297\, V_p + 673.64$$

By substituting $UCS = 0.0127 V_P - 1.7208$ and $\rho = 0.6297\, V_p + 673.64$ and rearranging the equation gives;

$$q_{ult} = V_p(0.13 + 66.13D + 21.01B) + 70{,}711.99D + 22{,}465.89B - 17.36 \quad (16)$$

Table 5 presents a summary of the equations for two different types of sedimentary rock.

Table 5
Ultimate bearing capacity (q_{ult}) values for argillaceous and arenaceous rocks in this study

Sedimentary rocks	Ultimate bearing capacity (q_{ult})
Argillaceous rock	$q_{ult} = V_p(0.13 + 66.13D + 21.01B) + 70{,}711.99D + 22{,}465.89B - 17.36$
Arenaceous rock	$q_{ult} = V_p(0.24 + 17.11D + 5.44B) + 207{,}683.15D + 65{,}982.98B - 330.67$

As a result, an empirical bearing capacity equation has great importance during the early stages of engineering design works. Moreover, it can predict the geotechnical parameters at the site when information on the P-wave velocity is available. Thus, it is a practical way compared with extensive experimental works. Determination of rock samples via UCT laboratory tests is complicated, expensive, and time-consuming. Furthermore, it requires fresh core specimens. Therefore, some attempts have recently been made to develop indirect methods which use P-wave velocity values from the seismic refraction method.

6. Conclusion

Significantly, the utility and efficiency of this empirical equation for the bearing capacity of arenaceous and argillaceous rocks have been demonstrated. Furthermore, this proposed empirical equation derived from P-wave velocity can be considered an alternative method for designing shallow foundations. Basically, this study not only proves that the practical use and effectiveness of the empirical equation are valid but also highlights its ability to change the way we currently design foundations. By leveraging P-wave velocity as a key parameter, the proposed equation not only ensures accurate predictions of bearing capacity but also presents a sustainable alternative that aligns with the growing emphasis on cost-effectiveness and environmental consciousness in geotechnical engineering. The dual benefits of reduced costs and minimized environmental impact make this empirical equation a promising advancement for the field, showcasing its potential to streamline processes and contribute to more responsible and efficient geotechnical practices.

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