Incorporating suction in to the interpretation of plate load tests on unsaturated soils

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ABSTRACT

Results of plate load tests performed on saturated and unsaturated silty sand samples are presented. Very little is known about how to interpret the plate load test when performed on unsaturated soils comprising silt-sand mixtures, as the few published studies have been limited to either unsaturated clean sands or clays. The new plate load test results are interpreted using a bearing capacity equation, applicable when non-uniform χ_s profiles prevail in the soil, where χ_s is the contribution of suction to the effective stress. The interrelation between χ_s , strength and bearing capacity is shown. There is a good agreement between measured and computed bearing capacities. Assuming χ_s remains constant during a plate load test, compared to assumptions of constant suction or constant moisture content, simplifies interpretation without significant loss of accuracy. For a particular soil density, the bearing capacity for an unsaturated condition is found to be much larger than that for a fully saturated condition. It is also shown that the effect of suction on the bearing capacity is dependent on the initial location of the hydraulic state on the soil-water characteristic curve. Failing to account for suction and the hydraulic loading history may lead to incorrect interpretations. It is also demonstrated how plate load tests may be used to infer the value of χ_s at the soil surface.

Keywords: plate load test, bearing capacity, suction, hydraulic hysteresis

1 INTRODUCTION

The plate load test (PLT) is a widely used field test to determine a soil's ultimate bearing capacity and settlement under a given load. Many PLTs are performed on unsaturated soils. There is a need to understand the response of these tests to be able to evaluate or back-calculate the strength parameters of the ground properly so shallow foundations can be designed. Laboratory model footing tests (Oloo 1994; Mohamed and Vanapalli 2006; Oh and Vanapalli 2011) and in-situ PLTs (Consoli et al. 1998; Costa et al. 2003; Xu 2004) showed that the bearing capacity is significantly increased by suction in an unsaturated soil. The effects of suction may be accounted for using empirical correlations (Costa et al. 2003) or by considering it as an apparent cohesion (Vanapalli and Mohamed 2013; Vahedifard and Robinson 2016).

Hydraulic hysteresis is a phenomenon which gives rise to non-unique relationship between suction and degree of saturation in unsaturated soils. The suction value corresponding to a degree of saturation depends on whether the soil has undergone a drying or wetting process prior to achieving the current state. It has been found that the mechanical behaviour of unsaturated soil is influenced by the hydraulic hysteresis (e.g.

Nishimura and Fredlund 2002; Khalili and Zargarbashi 2010; Pedroso 2015). A recent study (Tang et al. 2017) also found this phenomenon to have significant influences on the bearing capacity. However, no interpretations of model footing test results and PLT results have considered how suction may have differing influences on bearing capacity depending on whether the soil is undergoing wetting or drying.

In this paper results of laboratory-controlled PLTs performed on saturated and unsaturated silty sand are presented. Different suction profiles are developed in unsaturated samples subjected to drying and wetting, by raising and lowering the water table. The suction profiles are determined by the measurements made using vibrating wire piezometers. The PLT results are interpreted using the effective stress concept. The influences of the hydraulic loading histories, soil densities, and suction profiles on the bearing capacities measured in the PLTs are systematically demonstrated.

2 PLATE LOAD TESTS ON UNSATURATED SILTY SAND

2.1 Test soil

The tested soil is decomposed granite sourced from the catchment area of Lyell dam, NSW, Australia

(referred to as Lyell silty sand). It is classified to be well-graded silty sand (SM) according to the Unified Soil Classification System (Yang and Russell 2015). The particle size distribution curve with a double logarithmic scale is shown in Fig. 1. It is approximately linear with a slope of 0.39. Linearity in the double logarithmic plane implies that the soil has a fractal distribution of particle sizes (Russell 2010), the fractal dimension of the particle size distribution is $D_s=3-0.39=2.61$. The specific gravity of Lyell silty sand is $G_s=2.55$. The maximum dry density ($\rho_{\max}$) is 2.02 g/cm^3 corresponding to the minimum void ratio ($e_{\min}$) of 0.26, and the minimum dry density ($\rho_{\min}$) is 1.51 g/cm^3 corresponding to the maximum void ratio ($e_{\max}$) of 0.69. The critical state friction angle of Lyell silty sand is $\phi'_{cs}=35.7^\circ$. The soil is cohesionless at the critical state, i.e., $c'=0 \text{ kPa}$.

Fig. 1. Particle size distribution for Lyell silty sand in a double logarithmic plane.

Fig. 2. Soil-water characteristic curve with hydraulic hysteresis and effective stress parameter plotted against suction.

The soil-water characteristic curve (SWCC) of Lyell silty sand is shown in Fig. 2. There is a main drying curve, a main wetting curve and an infinite number of connecting scanning curves in the double logarithmic s - S_r plane (Vo et al. 2016). Their mathematical expressions are:

$$S_r = \begin{cases} 1 & \text{for } \frac{s}{s_e} \leq 1 \\ \left(\frac{s}{s_e}\right)^{-0.52} & \text{for } \frac{s}{s_e} > 1 \end{cases} \quad (1)$$

$$S_r = \begin{cases} \left(\frac{s_{rd}}{s_{ae}}\right)^{-0.52} \left(\frac{s}{s_{rd}}\right)^{-0.2} & \text{for drying path reversal} \\ \left(\frac{s_{rw}}{s_{ex}}\right)^{-0.52} \left(\frac{s}{s_{rw}}\right)^{-0.2} & \text{for wetting path reversal} \end{cases} \quad (2)$$

where s is the suction, s_e is the suction value separating saturated from unsaturated states. $s_e = s_{ex}$ = the air expulsion suction which applies when the hydraulic state is on the main wetting curve. $s_e = s_{ae}$ = the air entry suction which applies when the hydraulic state is on the main drying curve. s_{rd} and s_{rw} are the points of suction reversal on the main drying and wetting curves, respectively. The slope of the main drying and wetting curves is -0.52 and the slope of scanning curves is -0.2. Linearity of the main drying and wetting curves in the double logarithmic s - S_r plane implies that the pore size distribution is fractal, and the fractal dimension can be inferred to be $D_p = -0.52 + 3 = 2.48$ (Russell 2014).

Samples with different initial void ratios were tested to take into account the void ratio (e) dependency of the SWCC. The e dependency can be captured in its entirety through s_{ae} and s_{ex} (Russell 2014):

$$s_{ae} = 1.5e^{-D_s} \text{ kPa} \quad (3)$$

$$s_{ex} = 0.05e^{-D_s} \text{ kPa} \quad (4)$$

The effective stress (σ') is (Bishop 1959):

$$\sigma' = \sigma_{net} + \chi s \quad (5)$$

where $\sigma_{net} = \sigma - p_a$ is the net stress, σ is the total stress and p_a is the pore air pressure. $s = p_a - p_w$ in which p_w is the pore water pressure. χ is the effective stress parameter (Vo et al. 2016):

$$\chi = \begin{cases} 1 & \text{for } \frac{s}{s_e} \leq 1 \\ \left(\frac{s}{s_e}\right)^{-0.55} & \text{for } \frac{s}{s_e} > 1 \end{cases} \quad (6)$$

$$\chi = \begin{cases} \left(\frac{s_{rd}}{s_{ae}}\right)^{-0.55} \left(\frac{s}{s_{rd}}\right)^{-0.21} & \text{for drying path reversal} \\ \left(\frac{s_{rw}}{s_{ex}}\right)^{-0.55} \left(\frac{s}{s_{rw}}\right)^{-0.21} & \text{for wetting path reversal} \end{cases} \quad (7)$$

The relationship between χ and s in a double log plane is shown in Figure 2. The dependency of χ on e is captured through s_{ae} and s_{ex} in Equations (3) and (4).

2.2 Testing apparatus

The testing apparatus consists mainly of a steel-framed testing rig, loading devices and a data acquisition system. The rig is 0.69 m wide by 2.07 m long by 1.14 m deep. The water table can be controlled in a sample in the rig by adding and draining water through a valve connected to the bottom of the rig. The front face of the testing rig is plastic plate. The details of the testing rig are shown in Fig. 3.

Fig. 3. The details of the testing rig.

The PLT equipment comprises a rigid circular steel plate, a load cell, a vertical displacement transducer and a worm gear actuator. The circular steel plate has a diameter d of 0.15 m. The vertical displacement is applied by a worm gear actuator. A vertical displacement transducer is used to convert the applied displacement into electrical signals. A load cell with a capacity of 45 kN is mounted to measure the load. The details of the loading devices are shown in Fig. 4.

Fig. 4. The details of the loading devices.

Three plate load tests can be conducted at different locations on the surface of one soil sample to check the repeatability of the tests results. The width of the testing rig and the distance between the centres of adjacent plate load tests is 0.69m. It is 4.6 times of the diameter of the plate ($d=0.15$ m) which is sufficient for boundary effects to be negligible and one test not to

interfere with the other.

2.3 Sample preparation and suction measurement

Before preparing a sample a layer of gravel with a thickness of 200 mm is placed at the base of the testing rig. Geotextile is then placed on top of the gravel layer to keep the silty sand separate from the gravel and to prevent the small silt and sand particles dropping into the voids of gravel. Silty sand samples are prepared using a combination of pluviation and vibration. The method involves making a soil sample in three layers. Each layer has a thickness of 160 mm and is placed using dry pluviation with a drop height of 500 mm. A uniform surcharge of 154 kPa is placed on the surface of the three layered sample and the entire testing rig is then vibrated using a mechanical vibrator with a frequency of 80 Hz. In forming a number of samples it is found that the thickness of each layer remains very similar after vibration indicating a uniform density.

Different densities may be achieved by varying the duration of vibration. Soil samples are prepared by vibrating for 3 mins, 6 mins and 9 mins, respectively, corresponding to void ratios of 0.39, 0.34 and 0.29 and relative densities of 0.69, 0.82 and 0.92.

After preparing the dry samples water is added through the valve connected to the testing rig near the bottom of a soil sample.

Some saturated samples are made unsaturated by subjecting them to a wetting process. This involves holding the water table constant at the same level as the geotextile separating the gravel layer from the silty sand sample. Capillary action then causes the water to move upwards into the voids of the silty sand sample.

A sample is made saturated by allowing the water level to rise to 20 mm above the sample surface and then allowing the sample to soak for 14 days. In trial tests the degree of saturation at different depths is measured and found to be larger than 0.96.

A sample is made unsaturated by subjecting it to a drying. This involves lowering the water table to the soil gravel interface. An unsaturated sample is assumed to have reached equilibrium when the changes of suction (measured in the samples) during a 24 hour period are less than 0.1 kPa.

Four vibrating wire piezometers are placed at three different depths to measure suction in the soil samples. Holes at different locations of the soil sample are created to insert the piezometers. The soil excavated from each hole is weighed and reduced by an amount that would occupy the volume of the piezometer. The soil is then placed back and compacted to the original density above the piezometer.

Two piezometers are inserted at a depth of 70 mm (about half of the diameter of the plate) to check the uniformity of the suction at a shallow depth. They also enable suction measurement of the soil close to the soil surface which has the greatest influence on the bearing capacity. Two other piezometers are placed at the

depths of 220 mm and 370 mm (1.5 times and 2.5 times the plate diameter) enabling the suction profile to be established.

2.4 Test procedure

A PLT involves positioning the plate on the soil surface and then loading it by imposing a displacement rate of 0.03 mm/s. A test is terminated when the failure load has been reached and the displacement is continued for a further 5-10 mm. The duration of the test is about 20 mins.

Three PLTs were performed at different locations on the surface of a sample: one test when the sample was saturated and two tests when the sample was unsaturated.

Subsamples may be taken from unsaturated samples immediately after a PLT is conducted. In this study they are taken using a cutting ring sampler (with an inner diameter of 63.2 mm and a height of 27 mm) at locations that are 60 mm from the piezometers and 100 mm from the test locations. The degrees of saturation are calculated and used to locate the hydraulic states on the SWCC.

3 INTERPRETATION OF THE RESULTS OF THE PLATE LOAD TESTS

3.1 Plate load test results

Each PLT has been assigned an identification number according to the relative densities of the soil samples. Figs. 5 to 10 show the load displacement curves obtained for the PLTs performed on saturated and unsaturated samples with different void ratios. The measured bearing capacities listed in Table 1 are defined as the peak load (shown on the figures) divided by the plate area.

Table 1. Measured and calculated bearing capacities for plated load tests.

Test number	Type	Measured bearing capacity (kPa)	Calculated bearing capacity (kPa)
SAT069	Saturated	70	64
USW069-1	Unsaturated wetting	500	361
USW069-2	Unsaturated wetting	463	361
SAT082	Saturated	101	84
USW082-1	Unsaturated wetting	544	447
USW082-2	Unsaturated wetting	562	447
SAT092	Saturated	132	108
USD092-1	Unsaturated drying	2546	2582
USD092-2	Unsaturated drying	2694	2582

In general it is indicated by the load displacement curves that the load increases with the applied

displacement to the peak value P_{max} and then drops or fluctuates slightly with further displacement. The two tests conducted under the same unsaturated condition at different locations on a sample produced (almost) identical load displacement curves and P_{max} values. For a soil sample with a certain relative density, P_{max} was larger for an unsaturated sample than a saturated sample. P_{max} increased with sample density in the tests on both saturated and unsaturated samples, although the samples had different suctions and this too has an influence on P_{max} .

Fig. 5. PLT results for saturated sample with void ratio of 0.39.

Fig. 6. PLT results and numerical simulation for unsaturated sample having undergone wetting with void ratio of 0.39.

Fig. 7. PLT results for saturated sample with void ratio of 0.34.

Fig. 8. PLT results and numerical simulation for unsaturated sample having undergone wetting with void ratio of 0.34.

Fig. 9. PLT results for saturated sample with void ratio of 0.29.

Fig. 10. PLT results and numerical simulation for unsaturated sample having undergone drying with void ratio of 0.29.

3.2 Initial locations of hydraulic states on the SWCC

For the unsaturated tests the suction and degree of saturation values at three depths (70 mm, 220 mm and 370 mm) measured by the piezometers are listed in Table 2. The measured S_r and s are plotted on the SWCC in Fig. 11, where the suction has been normalised by s_{ae} to capture the effects of the different void ratios of the samples.

An unsaturated sample which undergoes wetting prior to testing experiences an increase in its S_r . The corresponding movement of the hydraulic state is along the main wetting curve of the SWCC. An unsaturated sample which undergoes drying immediately prior to

testing, starting from a saturated condition, has a hydraulic state which moves along the top scanning curve of the SWCC. It is shown in Fig. 11 that the suction increase which occurred was not large enough for the hydraulic state to move from the top scanning curve on to the main drying curve.

Fig. 11. Plots of measured suction and degree of saturation on a normalized SWCC.

Table 2. suction and degree of saturation at different depths of the unsaturated soil samples and the calculated χ_s .

Test number	Depth (m)	Degree of saturation	Measured suction (kPa)	Calculated χ_s (kPa)
USW069-1 and USW069-2	0.07	0.27	7.9, 7.8	1.87
	0.15	0.26	-	-
	0.22	0.27	7.2	1.81
	0.30	0.29	-	-
USW082-1 and USW082-2	0.37	0.53	2.3	1.08
	0.07	0.48	5, 4.8	1.89
	0.15	0.52	-	-
	0.22	0.51	4.6	1.84
USD092-1 and USD092-2	0.30	0.56	-	-
	0.37	0.74	1	0.93
	0.07	0.37	23.4, 22.9	12.4
	0.15	0.41	-	-
USD092-2	0.22	0.42	21.1	11.6
	0.30	0.46	-	-
	0.37	0.95	5.6	4.1

3.3 Effect of suction on the bearing capacity

As shown in Table 1, the test results indicate that suction has a pronounced effect on the bearing capacity. Compared with the fully saturated conditions, the bearing capacity is increased by about 6 times, 4.5 times and 19 times due to suction for the soil samples with void ratios of 0.39, 0.34 and 0.29, respectively.

The influence of suction on the effective stress and thus the bearing capacity is taken into account through χ_s . It is important to know the way χ_s varies with depth (the χ_s profile). The values of χ_s at different depths are listed in Table 2 (the influence of hydraulic state locations is considered). In all unsaturated samples it was found that χ_s decreases slightly with depth within the shallowest parts of the samples. The reduction of χ_s with depth is more significant in the deepest parts of the

samples. Bilinear χ_s profiles are assumed here that best fit these experimental measurements. z is used to denote depth (in meters) in a sample. These bilinear χ_s profiles are plotted in Figs. 12 to 14 together with the χ_s values calculated based on the measured s .

Fig. 12. Assumed χ_s profile for unsaturated sample having undergone wetting with void ratio of 0.39.

Fig. 13. Assumed χ_s profile for unsaturated sample having undergone wetting with void ratio of 0.34.

Fig. 14. Assumed χ_s profile for unsaturated sample having undergone drying with void ratio of 0.29.

3.4 Numerical modelling of the PLTs

A series of finite element analyses are performed using the fully coupled flow deformation model for the bearing capacity problem presented by Tang et al. (2017). The soil was assumed to be elastic perfectly plastic and obey the Mohr-Coulomb failure criterion.

The plate is assumed to be rigid and rough. Hydraulic hysteresis and the influence of volume change and e on the SWCC and χ_s were considered.

The input parameters used to generate the numerical results are listed in Table 3. The elastic moduli, E , are calculated based on the load-displacement curves obtained by the plate load tests. Here the peak friction angle (ϕ'_p) is assumed to be equal to 38° , 39° and 40° for soil samples with initial D_r of 0.69, 0.82 and 0.92, respectively, each above the critical state value of 35.7° and increasing with soil density (Bolton 1986; Maeda and Miura 1999; Salgado et al. 2000).

Table 3. Model parameters used in the interpretations and numerical simulations of the plate load tests.

Test number	E (kPa)	ϕ'_p ($^\circ$)	s_{ae} (kPa)	s_{ex} (kPa)
SAT069	500	38	-	-
USW069-1 and USW069-2	3400	38	17.5	0.58
SAT082	600	39	-	-
USW082-1 and USW082-2	2900	39	26	0.87
SAT092	900	40	-	-
USD092-1 and USD092-2	17000	40	37.3	1.24

The duration of the PLT is sufficiently short for the moisture to stay constant in the samples during testing due to the silty sand having a relatively low permeability (Yang et al. 2014). To investigate how sensitive the bearing capacity is to the drainage conditions, a constant moisture content (ω) and constant χ_s condition were assumed in generating the numerical results. Three sets of numerical results (for rigid and rough circular plates) are compared to experimental data in Figs. 6, 8 and 10. It is shown that considering a wide range of initial densities and hydraulic state locations, numerical result agree well with experimental data and the errors associated with the constant χ_s assumption are less than 5% for Lyell silty sand. A constant χ_s condition can be assumed in calculation of bearing capacity without loss of significant accuracy.

The numerical analysis was also used to confirm that the 0.69 m wide sample and 0.69 m test location spacing was sufficient for boundary effects and interferences to be negligible (less than 1%).

3.5 Interpretation using bearing capacity theory

It is assumed that χ_s has a profile that varies linearly with depth according to:

$$\chi_s = (\chi_s)_0 + K_{\chi_s} z \quad (8)$$

where $(\chi_s)_0$ is the value of χ_s at the ground surface and $K_{\chi_s} = \partial(\chi_s)/\partial z$ is a constant. Vo and Russell (2016) analysed the bearing capacity problem using slip line theory and recognised that $c'\cot\phi'$ and χ_s have similar (and independent) effects on the governing equations.

The bearing capacity (q_u) of unsaturated soils may be expressed in dimensionless form as:

$$V = \frac{q_u}{c'_0 + (\chi s)_0 \tan \phi' + q_s \tan \phi'} \quad (9)$$

and is dependent on the dimensionless factor:

$$F = \frac{(K_c + K_{\chi s} \tan \phi')B + \gamma_t B \tan \phi'}{c'_0 + (\chi s)_0 \tan \phi' + q_s \tan \phi'} \quad (10)$$

along with the friction angle ϕ' . c'_0 is the cohesion at the soil surface, K_c is a constant defining the variation of cohesion with depth. γ_t is the total unit weight of the soil. B denotes the footing diameter and q_s is the surcharge applied to the surface of the soil adjacent to the footing.

The slip line theory is used to compute the unique relationships between V and F for different ϕ' values for rough circular surface footings. The relationships are presented in charts in Fig. 15. These relationships can be used to determine q_u for any combination of c'_0 , ϕ' , K_c , B , $(\chi s)_0$, $K_{\chi s}$, q_s and γ_t .

Note that for a soil that is saturated all the way to the soil surface, and when drained conditions prevail, $(\chi s)_0$ may be interchanged with $(-u_w)_0$, that is the negative of the pore water pressure at the ground surface, and $K_{\chi s}$ may be interchanged with $K_{uw} = \partial(-u_w)/\partial z$. For hydrostatic conditions $K_{uw} = -\gamma_w$.

Fig. 15. The unique relationships between N_c and N_γ for different ϕ' values, for rough circular footings.

Table 4. Values of N_c and N_γ for rough circular footings.

$\phi' (^{\circ})$	N_c	N_γ
5	8.058	0.08063
10	11.09	0.3224
15	15.85	0.9323
20	23.68	2.416
25	37.32*	6.073
30	62.72*	15.52
35	114.0*	41.88
40	228.5*	123.7
45	519.6*	417.7
50	1397.0*	1710.0

In Fig. 15, as $F \rightarrow 0$, $V \rightarrow N_c$. Also, as $F \rightarrow +\infty$, $V \rightarrow N_\gamma F / (2 \tan \phi')$ (N_c and N_γ are bearing capacity factors).

These limiting conditions are represented by the straight dash lines. Values of N_c and N_γ (Martin 2004) are given in Table 4 for rough circular footings.

When these values are incorporated in to a bearing capacity equation of the form:

$$q_u = (c'_0 + (\chi s)_0 \tan \phi' + q_s \tan \phi') N_c + \frac{1}{2} (K_c \cot \phi' + K_{\chi s} + \gamma_t) B N_\gamma \quad (11)$$

the q_u values obtained lie below those that would be obtained using the more exact slip line theory, as demonstrated in Fig. 15 for $\phi' = 30^\circ$. The difference is due to the superposition used in the bearing capacity equation making it approximate and conservative when estimating q_u (e.g. Griffiths 1982; Martin 2004).

Fig. 16. Comparison between the measured bearing capacities and values calculated using Equation (12).

Lyell silty sand is cohesionless, no surcharge was applied in the experiments, and the surcharge induced by the plate's penetration depth is negligible, so Equation (11) can be simplified to:

$$q_u = ((\chi s)_0 \tan \phi') N_c + \frac{1}{2} (K_{\chi s} + \gamma_t) B N_\gamma \quad (12)$$

The bearing capacity factors N_c and N_γ are dependent on ϕ' . The peak friction angles in Table 3 are used in the calculation. The profiles of χs in the experiments conducted here are bilinear. Only the χs profiles at shallow depths need to be considered since Poulos and Davis (1974) found that the stress in the soil, due to the load applied to a shallow footing, is concentrated within a depth of $1.5B$ from the footing. The q_u values are calculated using Equation (12) assuming a rough plate and are listed in Table 1.

For a given initial D_r the q_u for an unsaturated condition is clearly larger than that for a saturated condition. This is due to the influences of both $(\chi s)_0$ and $K_{\chi s}$ in the calculation (notice that $K_{uw} = -\gamma_w = -9.8 \text{ kN/m}^3$ for the saturated condition thus the unit weight component of the bearing capacity calculation is significantly reduced).

Fig. 16 compares the computed q_u with the measured values for both saturated and unsaturated conditions. It is shown that, generally, the computed q_u lie below the measured q_u and the errors associated with Equation (12) are less than 30%. The differences may be partly due to the assumptions used to compute N_c and N_γ (Martin 2004) and the superposition used in the bearing capacity equation.

4 CONCLUSIONS

The results of PLTs performed on the surface of saturated and unsaturated silty sand samples subjected to different densities and suctions are presented in this paper. Different suction profiles were developed in unsaturated samples which underwent drying and wetting processes. The initial locations of hydraulic states on the SWCC were determined through measurements of suction and degree of saturation.

The χ_s profiles were found to be bilinear. The parameters defining the upper part of the bilinear χ_s profile was fed in to a bearing capacity equation to obtain an approximate value of the bearing capacity, as long as the upper part of the bilinear profile represents the χ_s values within the zone of soil that most heavily influence a PLT. Fully coupled finite element analyses show that the values of χ_s may be assumed constant and equal to the initial values during plate loading and the associated errors is less than 5%.

It is observed that the bearing capacity calculated using the bearing capacity equation is slightly lower than those obtained from more exact analyses using slip line theory or finite element analysis. The difference arises from the superposition principal used to establish the bearing capacity equation and the assumption that the χ_s values remain constant during loading. The bearing capacities calculated by the equation are in reasonable agreement to the values measured in the PLTs, with differences less than 30%.

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