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Estimating the allowable bearing pressure of soils from small-scale plate load and direct shear tests: laboratory-prepared mixtures and natural subgrade soils

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Research Article

Keywords: Small-scale plate load test, Direct shear test, Allowable bearing pressure, Subgrade, Standard penetration test, Shallow foundation

Posted Date: November 3rd, 2026

DOI: https://doi.org/10.21203/rs.3.rs-11588929/v1

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Additional Declarations: No competing interests reported.

Abstract

The plate load test (PLT) is the most direct way of obtaining the allowable bearing pressure of foundation and subgrade soils, but full-scale testing is costly and slow. This study investigates whether a small-scale plate load test (SPLT) on remoulded soil, combined with direct shear testing (DST), can provide a reliable and conservative estimate of allowable bearing pressure. Six laboratory-prepared mixtures (sand–kaolin and sand–kaolin–aggregate) and five natural soils from sites in Pune, India, were tested. Soil was compacted in steel containers at maximum dry density or at field density, and plates of 0.10 m × 0.27 m and 0.20 m × 0.30 m were loaded using a large-box direct shear apparatus. The shear criterion was evaluated from DST parameters and the settlement criterion from the SPLT with Terzaghi–Peck scaling; the smaller value governed. Natural-soil results were compared with standard penetration test (SPT) values and one field PLT. The settlement criterion governed in five of six soils; Terzaghi net safe capacities from DST ranged from 451 to 1024 kN/m², whereas SPLT safe bearing pressures ranged from 200 to 850 kN/m². For the natural soils, the 0.20 m plate gave allowable pressures equal to 88–95% of SPT-based values, and the 0.10 m plate 62–82%. At a soft clayey-silt site, SPLT values of 29–36 kN/m² bracketed the field PLT value of 33 kN/m².

Combining SPLT (settlement) with DST (shear) gives conservative estimates of allowable bearing pressure where standard field equipment is unavailable, with the larger plate being less conservative.

Introduction

Foundations, abutments, retaining structures, embankments and pavement subgrades all ultimately rely on the soil beneath them. A shallow foundation must satisfy two conditions: it must be safe against shear failure of the supporting soil, and its settlement must remain within tolerable limits [1, 2]. The allowable bearing pressure is therefore the smaller of the net safe bearing capacity from the shear criterion and the safe bearing pressure from the settlement criterion. In granular soils, where undisturbed sampling is difficult, the settlement criterion often governs and in-situ tests are used for design [3, 4].

The plate load test (PLT) is regarded as the most reliable means of establishing bearing capacity and settlement behaviour because it loads the soil in a manner that resembles a foundation and needs no correlation [5]. Its use in site investigation is limited, however, by cost, duration and the very heavy gravity platform or reaction truss required to mobilise pressures close to the ultimate capacity. Alternative in-situ methods such as the standard penetration test (SPT) [6] are cheaper, but they rely on empirical correlations between blow count, friction angle and bearing pressure [7, 8, 9, 10].

Plate size matters. For clays, bearing capacity from shear considerations is almost independent of footing size, whereas in sands it increases with footing width, so tests with small plates tend to give conservative values in dense sand [5]. Scale effects on granular soils have been examined experimentally [11], the confining influence of a finite soil container on bearing capacity has been quantified [12], and load–settlement behaviour of footings in cohesionless soil has been compiled and

modelled [13]. Direct shear test (DST) results are in turn sensitive to specimen size and strain rate [14, 15].

These observations suggest a practical route to bearing-pressure estimation without a full-scale test: compact the soil in a rigid container at its field density, load a small plate with a modest, easily fabricated loading frame to obtain the settlement criterion, and obtain the shear criterion from DST parameters of the same soil. This approach has not been systematically checked against independent field estimates for a range of soil types. The objectives of this study were therefore (i) to carry out small-scale plate load tests on six laboratory-prepared sand–kaolin and sand–kaolin–aggregate mixtures with controlled composition and to compare the resulting bearing pressures with those from DST; (ii) to apply the same procedure to five natural soils, using two plate sizes, and to compare the allowable bearing pressures with those from SPT and, at one site, a field PLT; and (iii) to assess how conservative the proposed procedure is and where it is most useful in transportation-infrastructure geotechnics.

Theoretical background

The net ultimate bearing capacity q_{nu} is the gross ultimate capacity q_u less the overburden pressure at foundation level (Eq. 1). The net safe bearing capacity q_{ns} is obtained by applying a factor of safety F, taken as 3 in this study (Eq. 2). The safe bearing pressure q_p is the pressure that produces the permissible settlement, taken as 25 mm for a 1 m wide footing. The allowable bearing pressure q_a is the smaller of q_{ns} and q_p (Eq. 3).

$$q_{nu} = q_u - \gamma D_f$$

1

$$q_{ns} = \frac{q_{nu}}{F}$$

2

$$q_a = min(q_{ns}, q_p)$$

3

where γ is the unit weight of soil and D_f the depth of the foundation.

Bearing capacity from shear strength parameters

The shear criterion was evaluated with three equations. For a square footing, Terzaghi's equation [1] is written as Eq. (4), and for a rectangular footing of width B and length L as Eq. (5), where R_w is the water-

table correction (0.5 when the water table is at the foundation base), $q = \gamma D_f$, and N_c, N_q and N_γ are bearing capacity factors that depend on the friction angle φ.

$$q_u = 1.3cN_c + qN_q + 0.4\gamma BN_\gamma R_w$$

4

$$q_u = \left(1 + 0.3\frac{B}{L}\right)cN_c + qN_q + 0.5\left(1 - 0.2\frac{B}{L}\right)\gamma BN_\gamma R_w$$

5

Meyerhof's general equation [16] and the equation of IS 6403 [17] (based on the factors of Vesic [18]) take the form of Eq. (6), with shape factors *s*, depth factors *d* and, for IS 6403, a water-table factor *W'* applied to the third term; inclination factors are unity for vertical loading. In IS 6403 the net capacity uses $q(N_q - 1)$ in the second term.

$$q_u = cN_c s_c d_c + qN_q s_q d_q + 0.5\gamma BN_\gamma s_\gamma d_\gamma$$

6

Safe bearing pressure and scaling of plate results

The ultimate capacity of a plate, q_{up}, was identified from the load–settlement curve by the tangent-intersection method. For cohesionless soils the ultimate capacity of a footing of width B_f is scaled in proportion to width (Eq. 7), whereas for cohesive soils the capacity is independent of footing size ($q_{uf} = q_{up}$) [2, 5].

$$\frac{q_{uf}}{q_{up}} = \frac{B_f}{B_p}$$

7

For the settlement criterion, the permissible footing settlement S_f is converted to the corresponding plate settlement S_p using the relation of Terzaghi and Peck for cohesionless soils (Eq. 8, with widths in metres) and the linear relation for cohesive soils (Eq. 9) [2, 19]. The safe bearing pressure q_p is the plate pressure at S_p. For a 1 m footing with S_f = 25 mm, the permissible plate settlements are 2.64 mm (0.10 m plate) and 6.76 mm (0.20 m plate) in cohesionless soil, and 2.5 mm and 5.0 mm in cohesive soil.

$$\frac{S_f}{S_p} = \left[\frac{B_f(B_p + 0.3)}{B_p(B_f + 0.3)}\right]^2$$

8

$$\frac{S_f}{S_p} = \frac{B_f}{B_p}$$

9

Because the plate load test is of short duration, it captures only immediate settlement; for clays it cannot represent consolidation settlement, so the settlement result for cohesive soils must be interpreted with caution [5].

Materials and methods

Testing programme

Two series of tests were carried out. **Series A** used six laboratory-prepared mixtures so that the effect of fines content and coarse-particle replacement on bearing behaviour could be examined under controlled conditions. **Series B** used natural soils from five sites in Pune, India, and was intended to check the procedure against independent estimates from SPT and a field PLT. In both series the same soil was tested by standard Proctor compaction (Series A) or field-density determination (Series B), DST and SPLT.

Series A: laboratory-prepared mixtures

The mixtures were made from a uniformly graded sand, commercial kaolin and a coarse aggregate. The sand (IS 2720 Part 4 [20]) has D_{10} = 0.30 mm, D_{30} = 0.37 mm, D_{50} = 0.40 mm, D_{60} = 0.45 mm, a coefficient of uniformity of 1.50 and a coefficient of curvature of 1.01. The kaolin has a liquid limit of 43.72%, a plastic limit of 35.4%, a plasticity index of 8.32, a specific surface of 15 m²/g and an activity of 0.7. Three sand–kaolin mixtures (10, 30 and 50% kaolin) were prepared, and each was then modified by replacing 20% of its mass with aggregate while keeping the kaolin-to-sand ratio constant (8:72, 24:56 and 40:40 kaolin:sand). Soils 1 and 4 are cohesionless and Soils 2, 3, 5 and 6 are treated as cohesive (Table 1).

Standard Proctor compaction (IS 2720 Part 7) gave the maximum dry density (MDD) and optimum moisture content (OMC) of each mixture. DST was performed in the large-box apparatus on specimens compacted to the MDD, at normal stresses of 0.5, 1.0 and 2.0 kg/cm² (about 49, 98 and 196 kN/m²); the cohesion *c* and friction angle φ were obtained from the intercept and slope of the best-fit failure envelope (Table 1). The unit weight γ used in the bearing capacity calculations of Series A was taken equal to the MDD (for example 1.67 g/cm³ ≈ 16.7 kN/m³).

Table 1
Composition, compaction characteristics and direct shear parameters of the Series A soils

Soil	Kaolin : sand : aggregate (%)	Class	MDD (g/cm³)	OMC (%)	*c* (kN/m²)	φ (°)
Soil 1	10 : 90 : 0	Cohesionless	1.670	13.5	16.94	30.87
Soil 2	30 : 70 : 0	Cohesive	1.954	10.8	25.73	29.58
Soil 3	50 : 50 : 0	Cohesive	1.823	15.01	41.90	31.17
Soil 4	8 : 72 : 20	Cohesionless	1.930	9.0	13.65	35.53
Soil 5	24 : 56 : 20	Cohesive	2.160	7.6	39.90	30.04
Soil 6	40 : 40 : 20	Cohesive	2.035	12.0	48.65	30.75

MDD maximum dry density; OMC optimum moisture content

Series B: natural soils

Trial pits were excavated at five sites in Pune and the in-situ density and water content were determined at the pit bottom, by the core-cutter method (IS 2720 Part 29) for soft and cohesive soils and by sand replacement (IS 2720 Part 28) for hard or sandy soils [20]; two samples were taken at each site and averaged. Bulk soil from the pit bottom was taken to the laboratory for classification (IS 2720 Parts 2, 4 and 5), DST (IS 2720 Part 27) and SPLT. SPT was performed in accordance with IS 2131 [6] at four sites (Sites 1, 2, 3 and 5). At Site 4, a soft silt of high plasticity, a full-scale field PLT was performed in place of SPT. Soil properties are summarised in Table 2. The sites are classified as well-graded gravel–clayey gravel (Site 1), poorly graded sand (Sites 2 and 3), high-plasticity silt (Site 4) and well-graded sand (Site 5).

Table 2
Classification, field density and shear strength of the Series B soils

Site	Soil type	LL/PL/PI (%)	Bulk density (kN/m³)	*w* (%)	Dry density (kN/m³)	*c* (kN/m²)	φ (°)[a]	SPT *N*
Site 1	GW-GC	32.2/18.3/13.9	18.62	9.01	17.07	19.78	28.1	38
Site 2	SP	–	17.81	11.27	16.00	28.39	30.6	28
Site 3	SP	–	18.63	9.75	16.97	13.36	29.8	25
Site 4	MH	52.5/33.4/19.2	17.77	25.32	14.18	27.21	8.7	–
Site 5	SW	–	17.16	18.73	14.44	28.35	29.8	40

[a] Friction angle computed from the slope of the fitted shear stress–normal stress envelope. LL liquid limit; PL plastic limit; PI plasticity index; *w* water content; *N* SPT blow count for the second and third 15 cm of penetration

Small-scale plate load test

The SPLT apparatus (Fig. 1) uses the loading system of a large-box direct shear machine, with a lever arm and counterweight applying vertical load to a rigid steel plate resting on soil compacted in a steel container. Two containers were used: 278 × 278 × 200 mm (Series A, and Series B with the 0.10 m × 0.27 m plate) and 600 × 600 × 350 mm (Series B with the 0.20 m × 0.30 m plate). Plate settlement was measured with two dial gauges of 0.01 mm least count. Soil was compacted in the container at the MDD and OMC for Series A and at the field dry density and water content for Series B (Table 2).

The plate was placed at the edge of the container. A preliminary study on remoulded soil, using 5 × 5 cm, 10 × 10 cm and 10 × 27 cm plates positioned at the centre and at the edge, showed that the 10 × 27 cm plate at the edge produced the most conservative load–settlement response and the least confinement from the container walls (unpublished). The 10 × 27 cm plate was therefore adopted for Series A and, with a 20 × 30 cm plate, for Series B. Elastic calculations indicated that roughly 40% of the applied pressure reaches the base of the container, so that boundary effects were judged to have limited influence.

Load was applied in increments of one-fifth of the estimated ultimate capacity. For each increment the settlement was recorded at 1, 2, 4, 8, 15 and 30 min and thereafter hourly, and the next increment was applied after at least one hour or when the settlement rate dropped to 0.02 mm/min [5]. Loading continued to failure or to the capacity of the apparatus.

Evaluation of allowable bearing pressure

Footings of 1 m × 1 m and 1 m × 2 m were considered, founded at D_f = 1 m with the water table at foundation level, giving $R_w = W' = 0.5$. For each soil, q_{ns} (shear criterion) was computed from the DST parameters using the equations of Section 2.2, and q_p (settlement criterion) was read from the SPLT curve at the plate settlement corresponding to a 25 mm footing settlement. In addition, q_{ns} was computed directly from the SPLT ultimate capacity by Eq. (7) (Series A). The allowable bearing pressure was then taken as the minimum of the DST-based q_{ns} and the SPLT-based q_p (Eq. 3). For Series B, SPT-based allowable pressures were obtained from the average of the shear criterion (Terzaghi's equation with φ from the corrected *N*-value [8]) and the settlement criterion (IS 8009 [19], Peck–Hanson–Thornburn [8], Teng [9], Meyerhof [7] and Bowles [10]). For Series B, the footing size and depth assumed in the DST- and SPT-based calculations.

Results and discussion

Series A: load–settlement behaviour

Figure 2 shows the load–settlement curves of the six mixtures with the tangent construction used to identify q_{up} and the pressure at the permissible plate settlement. Plate ultimate capacities were 350, 400 and 600 kN/m² for the sand–kaolin mixtures with 10, 30 and 50% kaolin (Soils 1–3), and 460, 700 and 920 kN/m² for the corresponding mixtures with 20% aggregate (Soils 4–6). Replacing 20% of the mass with aggregate therefore raised the plate capacity by 31, 75 and 53% at kaolin-to-sand ratios of 1:9, 3:7 and 1:1, respectively. Aggregate replacement also raised the MDD from 1.67 to 1.93, from 1.954 to 2.16 and from 1.823 to 2.035 g/cm³, and lowered the OMC by 3 to 4.5 percentage points (Table 1), so the gain in capacity reflects both the denser packing and the coarser skeleton.

The safe bearing pressures at the permissible plate settlement were 220, 200 and 640 kN/m² for Soils 1–3, and 460, 850 and 360 kN/m² for Soils 4–6. These values do not increase monotonically with capacity: Soil 6, with the highest ultimate capacity, has a lower q_p than Soil 5 and also than Soil 3, because its curve is considerably softer (plate settlement reached about 18 mm by the end of loading in Soil 6, compared with about 14 mm in Soil 3). A high fines content with coarse inclusions evidently gives a stiff peak resistance but a compressible response at working pressure, which the settlement criterion captures and the shear criterion does not.

Series A: shear-criterion capacity from DST

The DST parameters (Table 1) show that the friction angle is nearly constant, at 29.6–31.2°, in five of the six mixtures, the exception being Soil 4 (35.5°), whereas cohesion rises with kaolin content (from 16.9 to 41.9 kN/m² without aggregate and from 13.7 to 48.7 kN/m² with aggregate). Table 3 gives the resulting net safe bearing capacities. For the 1 m × 1 m footing, Meyerhof's equation gives values 39–60% higher than Terzaghi's equation, and the IS 6403 equation gives values 11–14% higher. For the 1 m × 2 m footing, the Terzaghi values are 5–9% lower than for the square footing, as expected from the smaller

shape factors. Terzaghi's equation was used as the reference in the comparisons that follow because it is the most conservative of the three.

Table 3
Net safe bearing capacity q_{ns} (kN/m²) of the Series A soils from DST parameters, $F = 3$

Soil	1×1 m Terzaghi	1×1 m Meyerhof	1×1 m IS 6403	1×2 m Terzaghi	1×2 m Meyerhof	1×2 m IS 6403
Soil 1	451.42	646.09	508.03	420.62	539.81	445.40
Soil 2	556.94	771.64	623.47	513.64	645.56	542.00
Soil 3	924.19	1303.03	1027.84	841.79	1071.16	887.40
Soil 4	703.89	1125.19	801.49	670.46	867.47	712.74
Soil 5	825.41	1152.51	925.05	754.73	956.10	800.72
Soil 6	1024.28	1432.00	1137.04	931.65	1178.74	980.75

Footing founded at D_f = 1 m with the water table at the foundation base

Series A: allowable bearing pressure

Table 4 and Fig. 3 compare the shear- and settlement-based estimates. Two features stand out. First, the net safe capacity derived directly from the SPLT ultimate load by Eq. (7) is highly variable: for the cohesionless Soils 1 and 4 the width ratio of 10 inflates it to 1167 and 1533 kN/m², well above the DST values, while for the cohesive soils it falls between 133 and 307 kN/m², below the DST values. Direct extrapolation of the plate ultimate load to a 1 m footing is thus not dependable and was not used to define the allowable pressure. Second, the SPLT safe bearing pressure was lower than the DST net safe capacity in five of six soils, by factors of 1.4 to 2.8 (DST/SPLT), so the settlement criterion governed. Only in Soil 5 (q_p = 850 kN/m² against 825 kN/m²) did the shear criterion govern, and only marginally. The allowable pressures for the 1 m × 1 m footing are therefore 220, 200, 640, 460, 825 and 360 kN/m² for Soils 1–6, and the values for the 1 m × 2 m footing differ only for Soil 5 (755 kN/m²).

Table 4
Series A bearing capacities and pressures (kN/m²) for 1 m × 1 m and 1 m × 2 m footings

Soil	Plate q_u	q_{ns} from SPLT (Eq. 7)	q_p from SPLT	q_{ns} from DST 1×1 m[a]	q_a 1×1 m	q_{ns} from DST 1×2 m[a]	q_a 1×2 m	Governing criterion
Soil 1	350	1166.67	220	451.42	220	420.62	220	Settlement
Soil 2	400	133.33	200	556.94	200	513.64	200	Settlement
Soil 3	600	200.00	640	924.19	640	841.79	640	Settlement
Soil 4	460	1533.33	460	703.89	460	670.46	460	Settlement
Soil 5	700	233.33	850	825.41	825	754.73	755	Shear
Soil 6	920	306.67	360	1024.28	360	931.65	360	Settlement

[a] *Terzaghi's equation. q_a = min(q_{ns} from DST, q_p from SPLT). For the 1 m × 2 m footing, the same q_p was used because the plate-to-footing settlement ratio depends on the footing width (*Eq. 8*), which was kept at 1 m*

The prevalence of the settlement criterion agrees with the established view that in granular and mixed soils settlement, rather than shear failure, usually controls the allowable pressure [3, 5]. It also shows why DST alone can mislead: it would give 451–1024 kN/m² for these soils, 1.4 to 2.8 times the pressure that the plate settlement permits in five of the six cases.

Series B: natural soils

Load–settlement behaviour and safe bearing pressure

Figure 4 shows the SPLT curves for the four granular sites (Sites 1, 2, 3 and 5) with each plate, and for the soft Site 4 with both plates. The dashed lines mark the permissible plate settlements of Section 2.3 (2.64 and 6.76 mm for cohesionless soils; 2.5 and 5.0 mm for Site 4, treated as cohesive). The resulting safe bearing pressures are given in Table 5. The 0.20 m plate gives larger q_p than the 0.10 m plate at every site, by 13% (Site 2) to 43% (Site 5) for the granular soils and 24% for Site 4 (36 against 29 kN/m²). This is consistent with the conservative character of small plates noted in IS 1888 [5].

Allowable bearing pressure from SPLT and DST

The shear-criterion capacity from DST (Terzaghi's equation) was 395.6, 615.5, 277.2, 95.6 and 619.6 kN/m² for Sites 1–5. The allowable pressure from the 0.20 m plate and DST was governed by settlement

at Sites 2, 3, 4 and 5, and by shear at Site 1, where q_{ns} (395.6 kN/m²) was only slightly below q_p (410 kN/m²). With the 0.10 m plate, settlement governed at all five sites. The friction angle at Site 4 (8.7°) is much lower than at the granular sites (28–31°), reflecting the high water content (25.3%) and plasticity of this silt.

Table 5
Shear- and settlement-based bearing pressures (kN/m²) and allowable pressure of the Series B soils from DST and SPLT

Site	q_{ns} from DST	q_p SPLT 0.10 m	q_p SPLT 0.20 m	q_a (0.10 m plate)	q_a (0.20 m plate)
Site 1	395.56	320	410	320	395.56
Site 2	615.52	240	270	240	270
Site 3	277.18	225	260	225	260
Site 4	95.63	29	36	29	36
Site 5	619.60	245	350	245	350
$q_a = \min(q_{ns}$ from DST, q_p from SPLT)					

Comparison with SPT and field PLT

Table 6 and Fig. 5 compare the SPLT/DST allowable pressures with those from SPT, which ranged from 275 to 416 kN/m². At every SPT site the SPT-based settlement criterion governed (q_p of 275–416 kN/m² against q_{ns} of 383–691 kN/m²). The 0.20 m plate with DST gave allowable pressures equal to 88–95% of the SPT values (395.6 against 416, 270 against 308, 260 against 275, and 350 against 396 kN/m²), and the 0.10 m plate gave 62–82%. In every case the SPLT-based estimate was below the SPT-based estimate, so the procedure was conservative relative to SPT at all four sites. The difference was smallest for the larger plate, which therefore leads to less uneconomic designs, whereas the smaller plate, at 62% of SPT at Site 5, may be excessively conservative for dense or well-graded granular soil. The DST shear criterion did not track the SPT shear criterion consistently: it was lower than the SPT value at Sites 1 and 3 and higher at Sites 2 and 5, which is a reminder that both are empirical estimates with their own scatter [7, 8].

Table 6
Allowable bearing pressure (kN/m²) from SPT and from SPLT with DST for the Series B granular sites

Site	SPT q_{ns}	SPT q_p	SPT q_a	SPLT 0.20 m q_a	Ratio to SPT (%)	SPLT 0.10 m q_a	Ratio to SPT (%)
Site 1	690.73	416	416	395.56	95	320	77
Site 2	437.65	308	308	270	88	240	78
Site 3	382.66	275	275	260	95	225	82
Site 5	451.28	396	396	350	88	245	62

At Site 4, where SPT was not performed, a full-scale field PLT gave an ultimate capacity of 99 kN/m², which with $F = 3$ corresponds to a safe pressure of 33 kN/m. The SPLT allowable pressures, 29 kN/m² (0.10 m plate) and 36 kN/m² (0.20 m plate), correspond to 88% and 109% of this value, i.e. the two plate sizes bracket the field result. Given that Site 4 is a soft, high-plasticity silt with allowable pressures an order of magnitude below those of the granular sites, the agreement suggests that the procedure also captures the lower end of the range, although it rests on a single field test.

Implications for transportation Geotechnics and limitations

Earthworks, subgrades, abutment and retaining-wall footings, culvert foundations and airfield facilities frequently require bearing-pressure estimates at many locations, often where heavy PLT equipment cannot be mobilised. The SPLT/DST procedure needs only a bulk sample, a compaction container and a modest loading frame, and it yielded conservative estimates against SPT at all four granular sites and against a field PLT within about 10% at the soft site. It can therefore serve as a screening or preliminary-design tool, with the larger plate preferred where economy matters.

Several limitations apply. The tests were on remoulded soil compacted in a container, so natural structure, layering and saturation are not represented; the water table was assumed at foundation level in the calculations but the samples were tested at their field or optimum moisture content. The container is small relative to the stress bulb of a 1 m footing, and the extrapolation from plate to footing relies on empirical scaling that is uncertain for gravelly soil. The SPT-based values used as the benchmark are themselves empirical, each site was tested once, and only one field PLT was available. The ratios in Table 6 should therefore be treated as indicative. For these reasons, results should be corroborated by field tests before they are used for final design.

Conclusions

Small-scale plate load tests and direct shear tests on six laboratory-prepared mixtures and five natural soils support the following conclusions:

1. Replacing 20% of sand–kaolin mixtures by aggregate raised the plate ultimate capacity by 31–75% (from 350–600 to 460–920 kN/m²) and the maximum dry density by 0.2 g/cm³ on average, but it did not guarantee a higher safe bearing pressure: Soil 6 had the highest ultimate capacity but a low settlement-based pressure (360 kN/m²).
2. For the laboratory mixtures, the settlement criterion governed the allowable bearing pressure in five of six soils. The DST net safe capacity (Terzaghi) exceeded the SPLT safe bearing pressure by factors of 1.4–2.8, so DST alone is unconservative. Meyerhof's and IS 6403 equations gave 39–60% and 11–14% higher capacities than Terzaghi's equation, respectively.
3. Direct extrapolation of the plate ultimate load to footing capacity by width ratio was erratic (133–1533 kN/m²) and should not be used to define the allowable pressure.
4. For natural granular soils, the allowable pressure from the 0.20 m plate with DST equalled 88–95% of the SPT-based value, and that from the 0.10 m plate 62–82%. The procedure was therefore conservative at all sites, and the larger plate was less conservative.
5. At a soft high-plasticity silt, the 0.10 m and 0.20 m plates gave 29 and 36 kN/m², bracketing the field PLT safe pressure of 33 kN/m².
6. The recommended procedure is to determine q_{ns} by DST and q_p by SPLT on soil compacted at field density and moisture content, and to adopt the smaller value as the allowable bearing pressure, where standard field equipment is unavailable.

Future work should compare SPLT results on undisturbed samples with field PLT in the same pit, evaluate saturated conditions, vary plate position and dimension, and test more sites and soil types, including a larger set of field PLT benchmarks. A portable multi-purpose instrument capable of DST, CBR and SPLT has been designed for this purpose, and its performance against conventional equipment remains to be evaluated.

Declarations

Author Contribution

Kakasaheb D. Waghmare: Conceptualization, Methodology, Formal analysis, Writing – original draft, Writing – review & editing, Project administration (corresponding author). Harsh Rajput: Investigation (preliminary small-scale plate load tests on remoulded soil, used to select plate size and position), Data curation. Archana: Investigation (laboratory tests on the sand–kaolin and sand–kaolin–aggregate mixtures), Data curation, Validation. Dipraj: Investigation (tests on the natural soils, including SPT and the field plate load test), Data curation, Validation. K. K. Tripathi: Supervision, Conceptualization, Resources, Writing – review & editing. All authors read and approved the final manuscript.

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Figures



Figure 1

Arrangement of the small-scale plate load test: soil container on the base of the large-box direct shear apparatus, reaction frame, lever arm, loading arm and counterweight

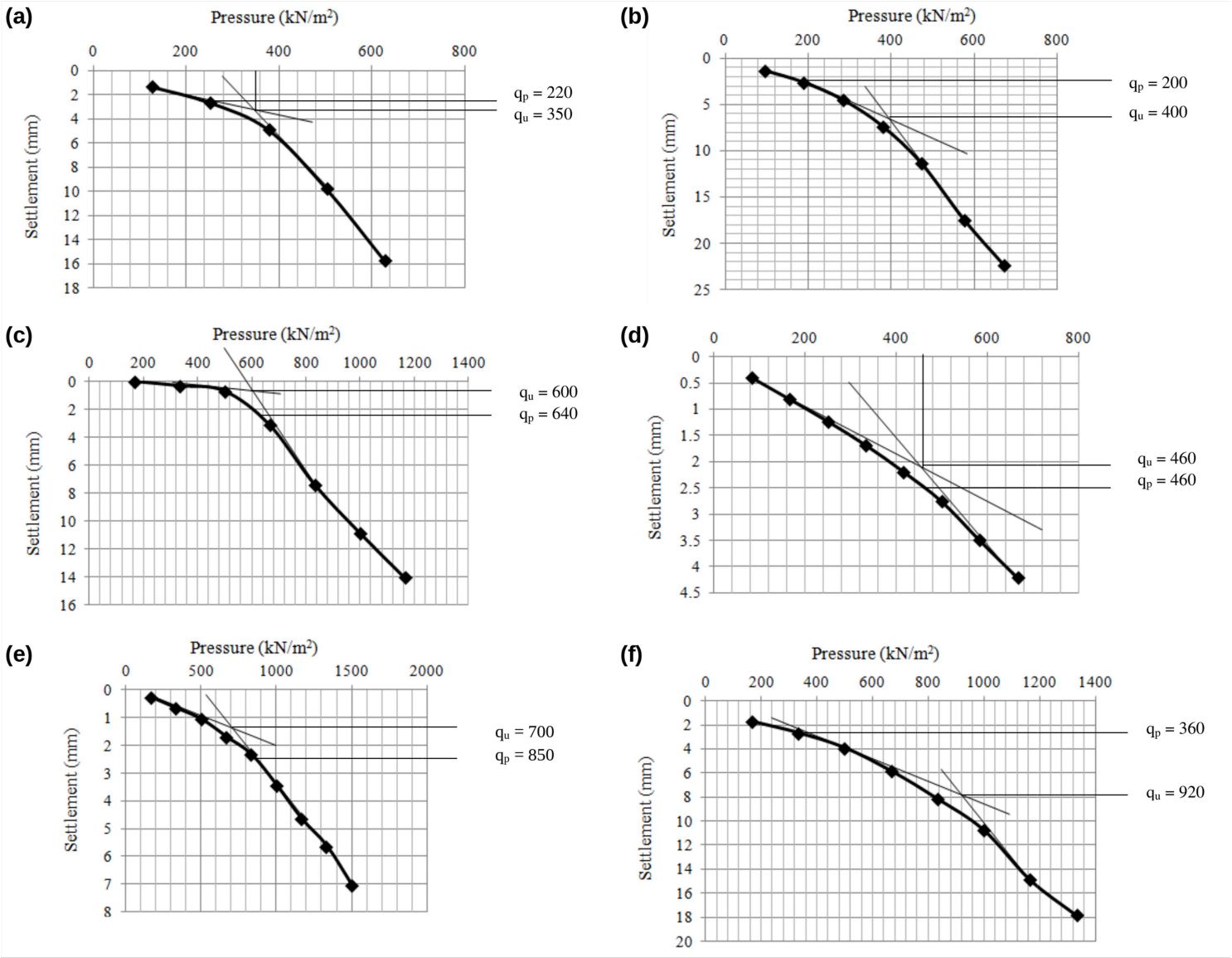


Figure 2

Load–settlement curves from small-scale plate load tests on a Soil 1, b Soil 2, c Soil 3, d Soil 4, e Soil 5 and f Soil 6, with the tangent-intersection construction; q_u is the plate ultimate capacity and q_p the pressure at the permissible plate settlement

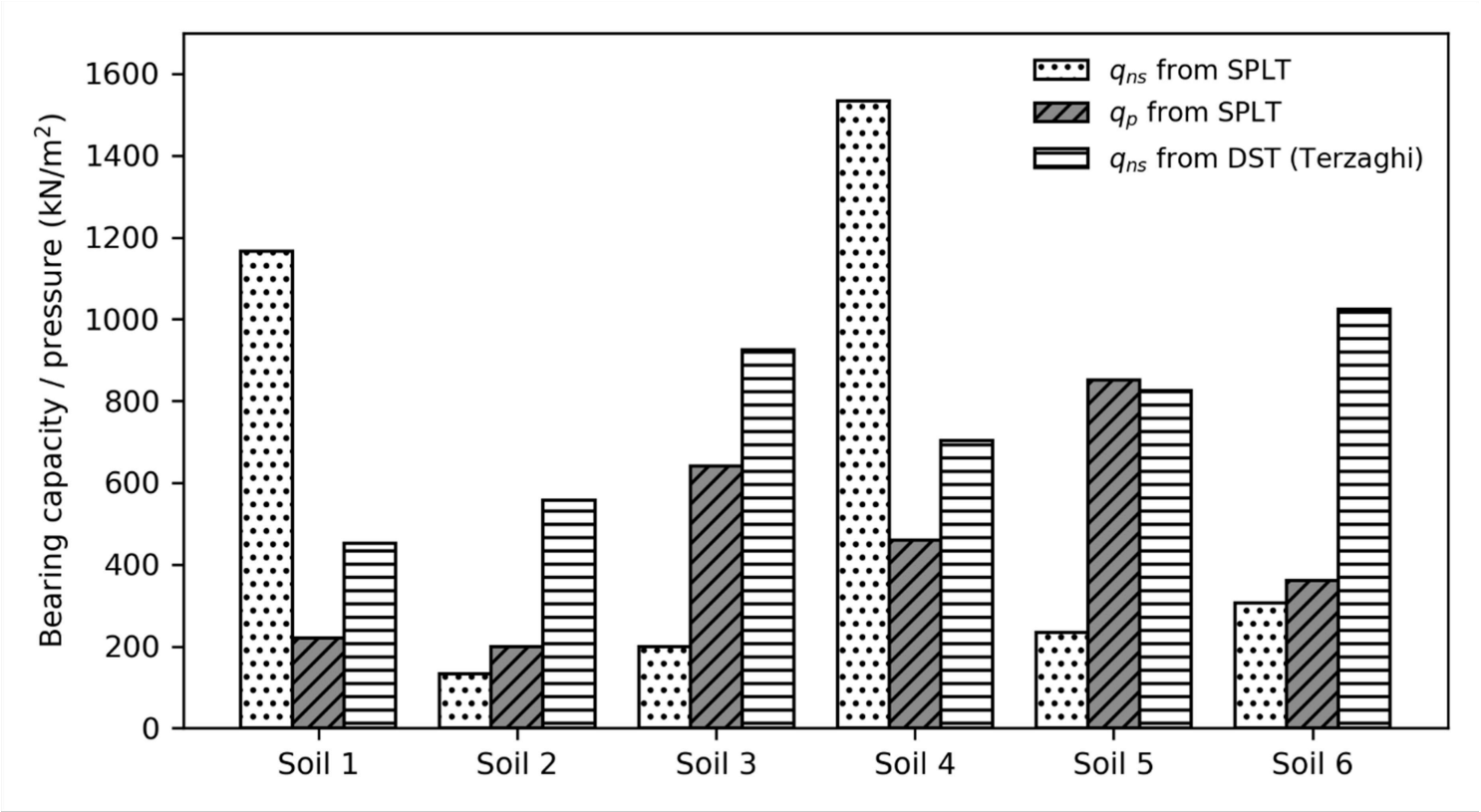


Figure 3

Comparison of net safe bearing capacity from the small-scale plate load test (SPLT) by Eq. (7), safe bearing pressure from SPLT, and net safe bearing capacity from DST for a 1 m × 1 m footing on the Series A soils

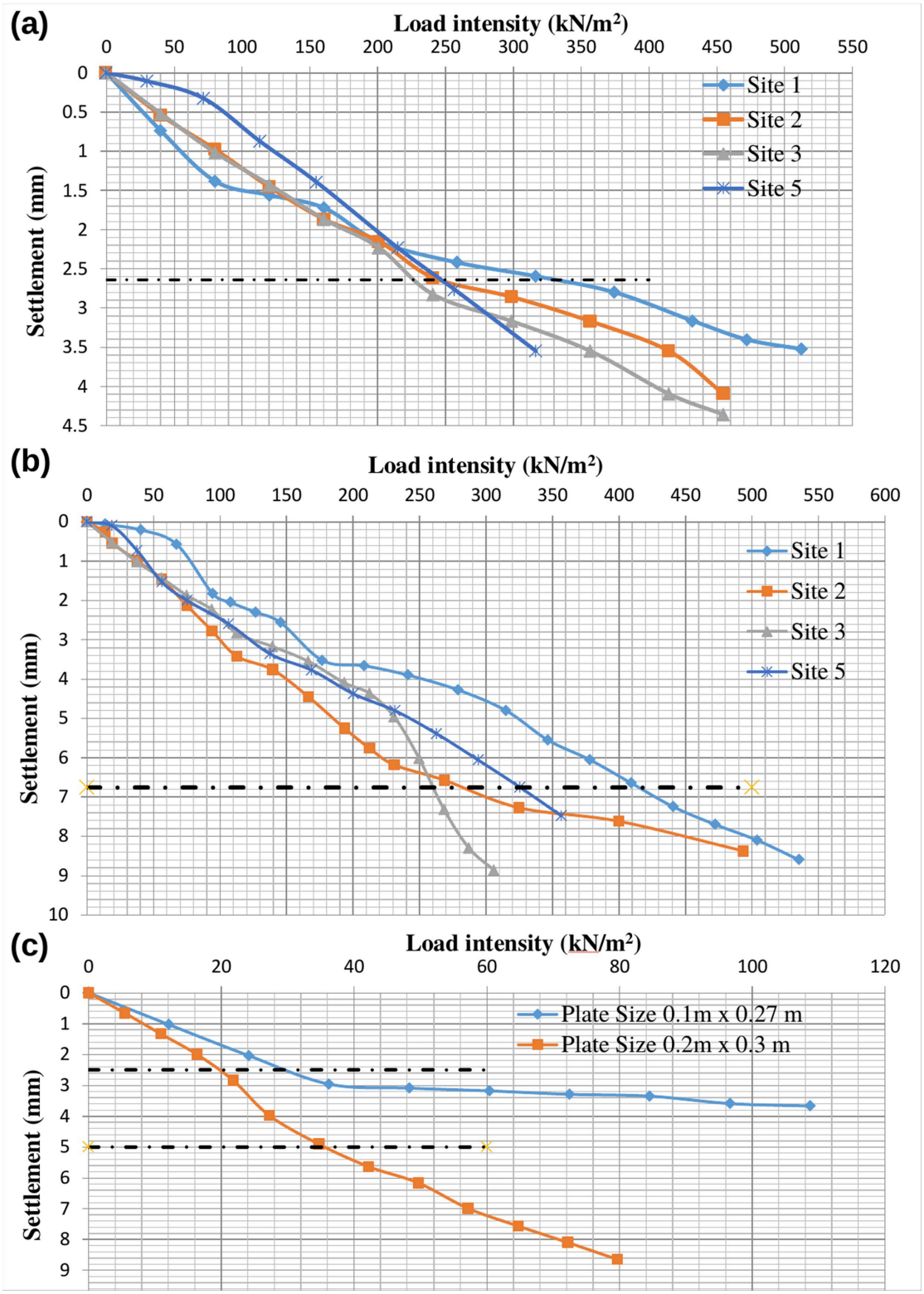


Figure 4

Load–settlement curves from small-scale plate load tests: a 0.10 m × 0.27 m plate on Sites 1, 2, 3 and 5; b 0.20 m × 0.30 m plate on Sites 1, 2, 3 and 5; c both plates on Site 4. Horizontal dash-dot lines show the permissible plate settlement

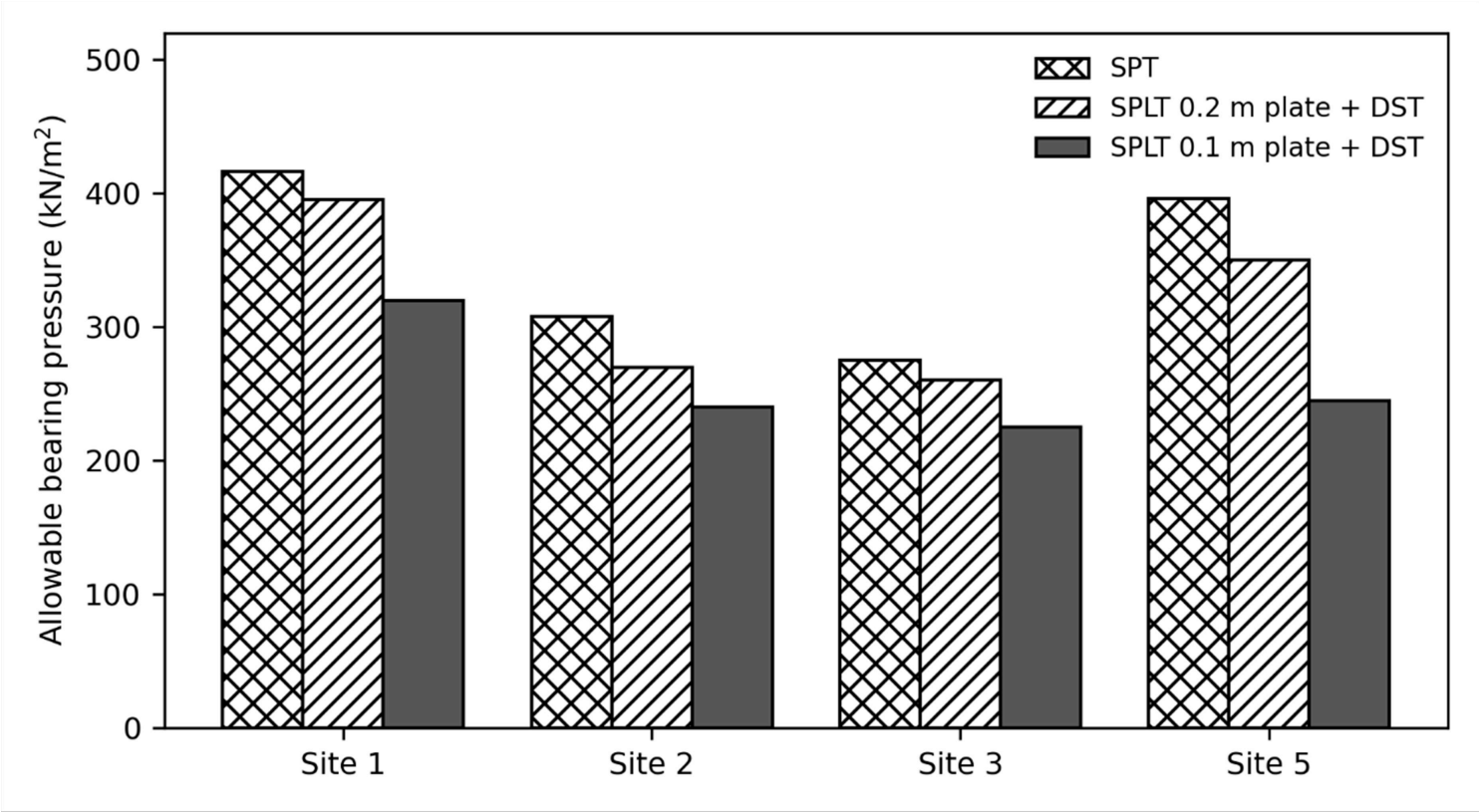


Figure 5

Allowable bearing pressure obtained from SPT, from SPLT with the 0.20 m plate and DST, and from SPLT with the 0.10 m plate and DST, for the four granular sites