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Are we overdesigning: Results of ISSMGE survey

Sommes-nous en surconception : résultats de l'enquête ISSMGE

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ABSTRACT: In June 2018, geotechnical engineers from the Asian Region of the ISSMGE asked: "Are we overdesigning?" In response, the ISSMGE initiated a survey involving ten simple geotechnical problems on clays or sands, based on real-life geotechnical data from the two National Geotechnical Experimentation Sites at Texas A&M University. With the assistance of the various regions of the ISSMGE, the survey was taken online in 2020 and 2021, and close to 240 responses were received. These responses are analysed and presented in this paper and compared with results of full-scale tests conducted on the Texas A&M sites, model calculations, and reliability-based solutions by TC 304. In almost all cases, the average of the responses gave a reasonable answer to the problem. However, the range of responses was unacceptably large and contained many highly improbable outcomes. Only two of the ten problems considered indicated a tendency towards overdesign. The remainder of the problems included evidence of both over- and underdesign. There is a clear need for the ISSMGE to focus on the implementation of existing knowledge and on improving the state of practice in the industry.

RÉSUMÉ : En Juin 2018, les ingénieurs géotechniciens de la région asiatique de la SIMSG ont demandé "est-ce que nous surconcevons ?". En réponse, la SIMSG a lancé une enquête impliquant dix problèmes géotechniques simples sur des argiles ou des sables, basés sur des données géotechniques réelles provenant des deux sites d'essai, Sites Nationaux d'Expérimentation Géotechnique a Texas A&M University. Avec l'aide des différentes régions de la SIMSG, l'enquête a été mise en ligne en 2020 et 2021 et près de 240 réponses ont été reçues. Ces réponses sont analysées et présentées dans cet article et comparées aux résultats des tests à grande échelle menés sur les sites Texas A&M, aux calculs de modèles et aux solutions basées sur la fiabilité par le TC 304. Dans presque tous les cas, la moyenne des réponses a donné un résultat raisonnable en réponse aux problems poses. Cependant, l'éventail des réponses était inacceptablement large et contenait de nombreux résultats hautement improbables. Seuls deux des 10 problèmes examinés indiquaient une tendance à la surconception. Le reste des problèmes comprenait des preuves de sur- et de sous-conception. Il est clairement nécessaire que la SIMSG se concentre sur la mise en œuvre des connaissances existantes et sur l'amélioration de l'état des pratiques dans l'industrie.

KEYWORDS: geotechnical design, overdesign, design calibration, practice survey.

1 INTRODUCTION

At the XVI Danube European Conference (Skopje Macedonia, June 2018), the question was asked: "Are we overdesigning?" Although the question originated from the Asian Region, it is valid internationally where different countries and users of various testing techniques or design codes tend to follow local practice for geotechnical design.

In response, the Corporate Associates Presidential Group (CAPG) of the ISSMGE conducted a survey of current geotechnical design practices worldwide. The survey was set up to assess the consistency of calculation models and design methods for a variety of geotechnical structures and, where possible, to compare the results with full-scale tests, model solutions and reliability analyses.

The survey was based on two soil profiles – one in clay and the other in sand. Soil test results, typical of those one would find in a geotechnical investigation report, were given for each soil profile. Excel spreadsheets were also provided, with numerical data to assist with data analysis. Unbeknown to the respondents, the data came from investigations conducted on two test sites on the campus of Texas A&M University at College Station, Texas, USA (Briaud 1997; Briaud 2021a; Briaud 2021b.)

Ten specimen problems were posed, including concentrically and eccentrically loaded spread footings, axially and laterally loaded piles, slopes and retaining structures. The idea was to keep the problems easy to analyse and representative of everyday geotechnical structures.

Some problems required prediction of the performance of the geotechnical structure (prediction problems) while others called for the design of the structure as it would be constructed (design problems). The prediction problems were aimed at assessing the selection of parameters and calculation models. The design problems were aimed at assessing the provisions made for safety and serviceability in geotechnical design. Respondents were asked to answer as many of the problems as they wished, preferably based on the work they do on a day-to-day basis. Partial solutions were also accepted.

Initially, respondents were requested to submit their responses by email in .pdf format. Later, due largely to the assistance received from the Young Members Presidential Group (YMPG), the survey was taken online.

2 GEOTECHNICAL DATA

2.1 *Data provided*

Although not disclosed at the time, the geotechnical data was from the National Geotechnical Experimentation Sites at Texas A&M University. The data was extracted from Briaud (1997), was digitised, and provided in both graphical and numerical format.

The information included moisture content, grading, SPT, CPT/CPTu and Pressuremeter test data at both sites. In addition, bulk density, dry density, over-consolidation and undrained shear strength data was provided for the clay site, and dilatometer and cross-hole seismic data for the sand site. The graphical representation of this data is given in Annexures A and B at the end of this paper.

The location of the sites, their geological history, site stratigraphy and soil properties are presented in Briaud (1997). The original survey and the numerical data for both sites can be found at original survey and soils data (scroll down to the overdesign survey files).

2.2 *Clay site*

The soils at the clay site are predominantly alluvial in origin and are over-consolidated. The profile can be summarised as follows:

0 – 5.5m	Very stiff clay
5.5 – 6.5m	Medium dense sand parting (intermittent)
6.5 – 12.0m	Very stiff clay
12.0m +	Highly weathered shale.

The water table is at a depth of about 6m below ground level.

2.3 *Sand site*

The soils at the sand site are also primarily of alluvial origin and the profile is as follows:

0 – 4.0m	Silty sand
4.0 – 8.0m	Clean sand
8.0 – 12.5m	Clayey sand
12.5m +	Highly weathered shale.

The water table is at a depth of about 5m below ground level.

3 ANALYSIS OF DATA

As will be seen from the discussion of the ten sample problems, the answers varied over a wide range. Some answers were clearly incorrect, and some were frivolous. These answers were omitted from the analysis of the data. However, outliers (very high or very low values) still existed within the remaining data. These outliers distort the average values and skew the analysis. Extreme outliers were also omitted, but values that had the appearance of a considered response (e.g. accompanied by input parameters, calculation methods, etc.) were retained in the data set.

The elimination of data is a subjective process. To make clear where high or low values have been removed, the range of values considered in each analysis is shown on the figures where appropriate. Anyone interested in re-analysing the complete data set can retrieve the original data from overdesign survey combined responses. This Excel file includes all individual responses to the problems, complete with parameters used, comments and references.

4 DETAILS OF SURVEY RESPONDENTS

The first section of the survey gathered general information about the respondents.

Two hundred and thirty-six responses were received from 39 countries. The number of responses received from the top ten countries is shown in Figure 1. The occupation and experience of the respondents are shown in Figures 2 and 3. The design methods used (WLD – working load design, LRFD – load and resistance factor design, PF LSD – partial factor limit states design, or Other) are shown in Figure 4.

Two additional questions were asked regarding the design codes used in the respondent's country and the respondent's favourite textbooks. These responses are not analysed in this paper. They can, however, be viewed by downloading the original data from the website given above.

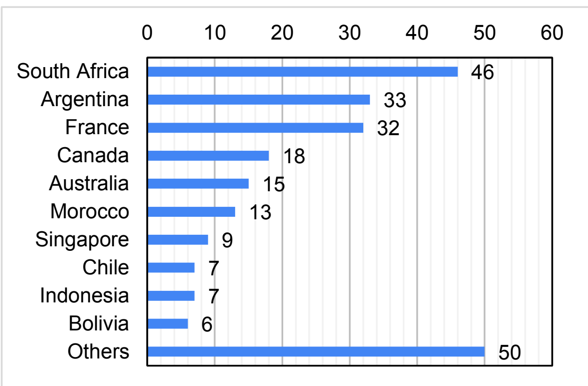
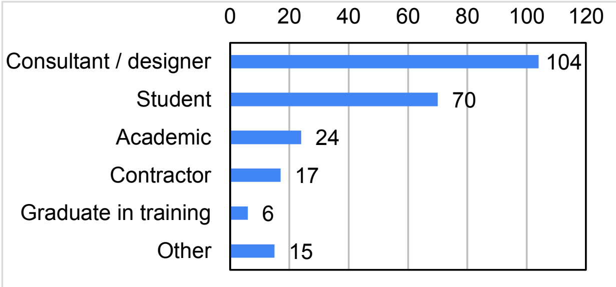


Figure 1. Number of responses from top ten countries.

Figure 2. Occupation of respondents.



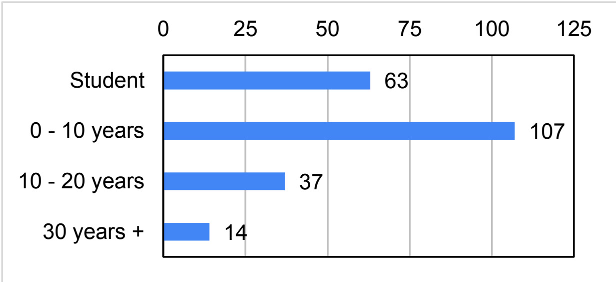


Figure 3. Respondents' years of experience.

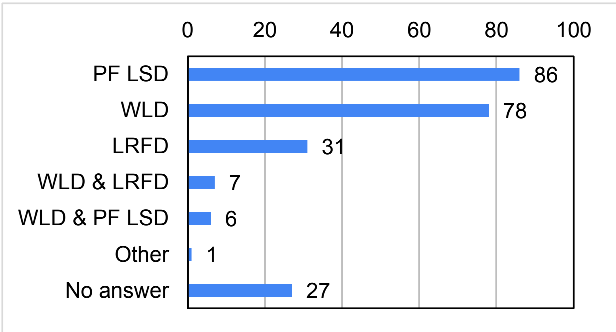


Figure 4. Design methods used in respondent's country.

In addition to the submissions by survey respondents, ISSMGE Technical Committee 304 (TC 304: Engineering Practice of Risk Assessment and Management) compiled a report providing probabilistic analyses of the survey questions (Ching & Zhang 2020). This report can be downloaded from TC 304 report.

5 CLAY 1: VERTICALLY LOADED STRIP FOOTING

5.1 *The problem (Figure 5)*

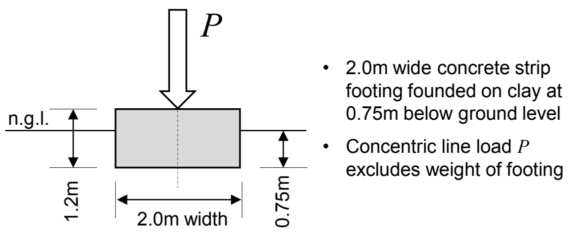


Figure 5. CLAY 1 – strip footing on clay.

CLAY 1 is a prediction problem in which respondents were asked to predict the applied load that will cause bearing capacity failure of the footing (P_{ult}) and the applied load that will cause the footing to settle 25mm in the long term (P_{25mm}).

5.2 *Responses*

The predicted bearing capacity (P_{ult}) is given in Figure 6 and the load that will cause 25mm settlement (P_{25mm}) is given in Figure 7.

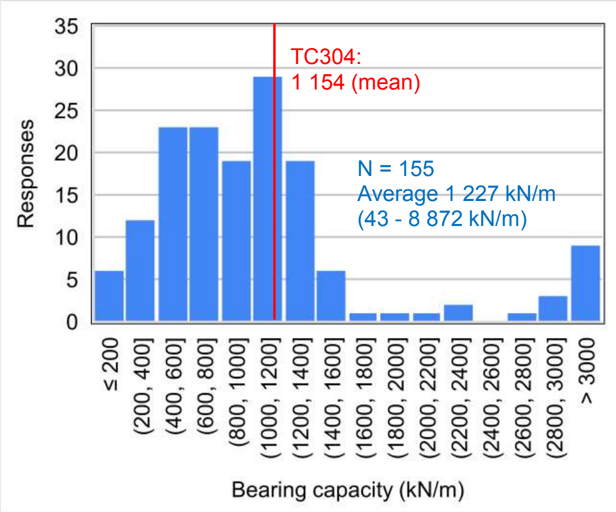


Figure 6. Predicted bearing capacity (P_{ult}).

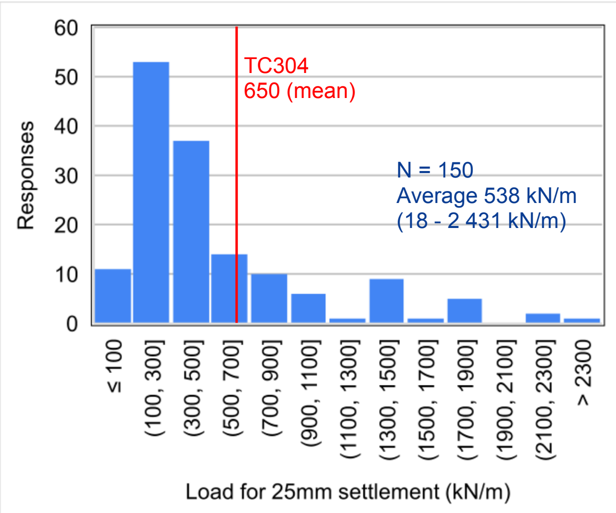


Figure 7. Predicted load for 25mm settlement (P_{25mm}).

5.3 *Analysis of responses*

Although the mean values may be reasonably acceptable, it is clear from Figures 6 and 7 that there is a considerable range of results. The probabilistic solutions prepared by TC 304 (Yang et al. 2020) show that it is highly improbable that the bearing capacity of the footing will exceed 3 000 kN/m or the load for 25mm settlement will exceed 2 000 kN/m.

To assess whether the wide range of answers can be attributed to inexperience of some respondents, 30% of whom were still students, the results were sorted according to the experience of the respondents as shown in Figure 8. The average values for each group are not unreasonable, but there were more outliers in the less experienced categories. This is not necessarily so for the remaining problems.

Of the respondents who stated which method of analysis was used, approximately 70% used an undrained analysis (total stress), 26% used a drained analysis (effective stress) and 5% used direct correlations with in situ tests. The average prediction of the drained bearing capacity (2 081 kN/m) was more than double the average prediction of undrained bearing capacity (968 kN/m). Note that most of the high predictions (above 3 000 kN/m) were from drained analyses. Direct correlations with in situ tests gave an average prediction of 1 027 kN/m.

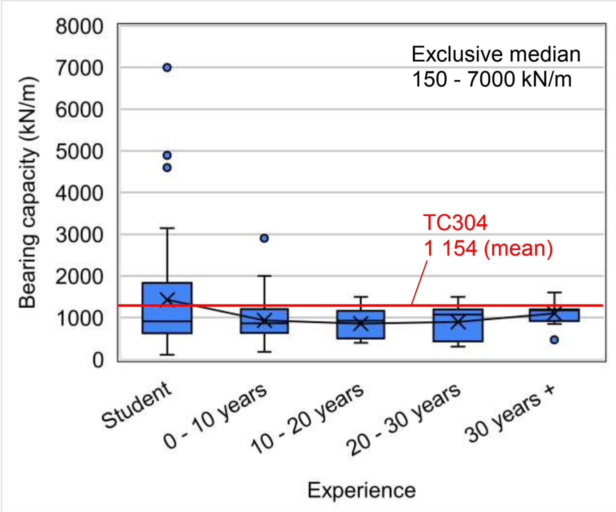


Figure 8. Bearing capacity (P_{ult}) v. experience.

Figure 9 was compiled to check if the wide range of predictions is attributable to the estimation of shear strength from the data provided or the calculation of bearing capacity using the estimated shear strength. This figure represents the results from the undrained (total stress) analyses. The blue dots are the survey responses. The solid black line in Figure 9 is the result of a Skempton-type bearing capacity calculation ($q = c\ N_c + p_0$) with a bearing capacity factor $N_c = 5.7$ (for z/B = 0.375), a bulk density of 21.5 kN/m^3 for the clay and a density of 24 kN/m^3 for concrete. The red cross is the result of the reliability analysis by TC 304 (Yang et al. 2020) based on an average undrained shear strength of 111.3 kPa, a coefficient of variation of 26.6% and a log-normal distribution.

Two conclusions can be drawn from Figure 9. Firstly, there is a large range of inferred undrained shear strengths for the clay (50 kPa – 200 kPa). The low values are probably a reflection of the lower bound to the results from triaxial tests on the clay. The mid-range values are a reasonable interpretation of all the triaxial test results and of the in situ test data. The justification for the high values (approaching 200 kPa) is unclear. The second conclusion is that the bearing capacities calculated for any given undrained shear strength show a large variation and deviate significantly from the results of traditional bearing capacity theory. The low values may be the result of the inclusion of a margin of safety, but the reason for the high values is unclear.

There is insufficient data to permit a similar analysis of the drained (effective stress) results.

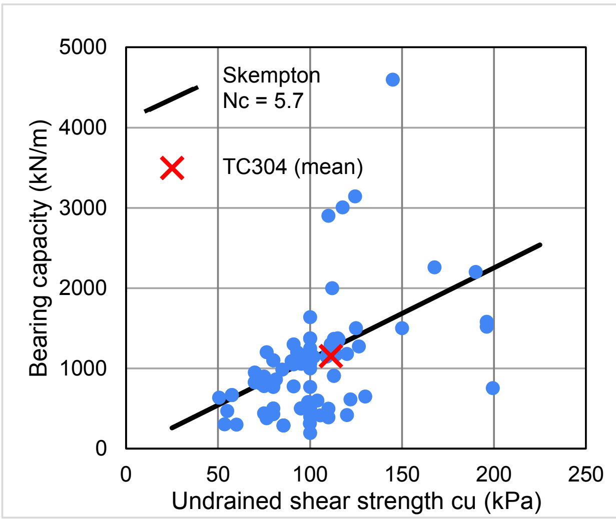


Figure 9. Comparison of results for varying values of shear strength.

No direct measurement of the compressibility of the clay profile is available. The stiffness of the clay must therefore be determined from in situ test results. For a drained elastic modulus profile varying from 10 MPa at founding level to 24 MPa at a depth of 12m, as may be inferred from the SPT N-values interpreted using Stroud's method (Stroud 1989), a simple elastic settlement analysis gives the applied load under which 25mm settlement will occur as 210 kN/m. This agrees reasonably with the deterministic value of 261 kN/m calculated by Yang et al. (2020) based on CPT and Pressuremeter test results. The survey responses shown in Figure 7 (average value 538 kN/m) are closer to the result of the probabilistic analysis by Yang et al. (2020) which gave a mean value of 650 kN/m.

6 CLAY 2: AXIALLY LOADED PILE

6.1 *The problem (Figure 10)*

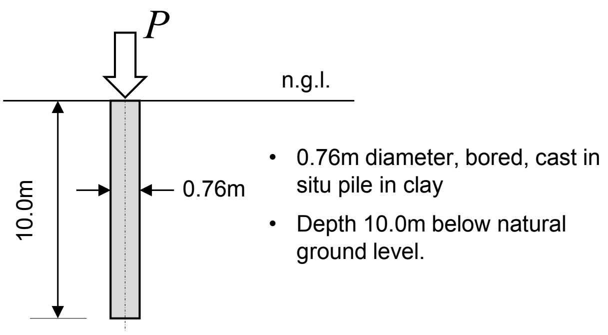


Figure 10. CLAY 2 – bored pile in clay.

CLAY 2 is a prediction problem in which respondents were asked to predict the ultimate capacity of the pile, the end bearing and shaft resistance (if calculated separately), and the load-settlement curve up to 80% of the failure load.

6.2 *Responses*

The predicted pile capacity (P_{ult}) is given in Figure 11, the ratio of end bearing to total pile capacity (P_b/P_{ult}) in Figure 12, and the load-settlement curves in Figure 13.

The TC 304 probabilistic assessment is given in Shuku and Hachich (2020a).

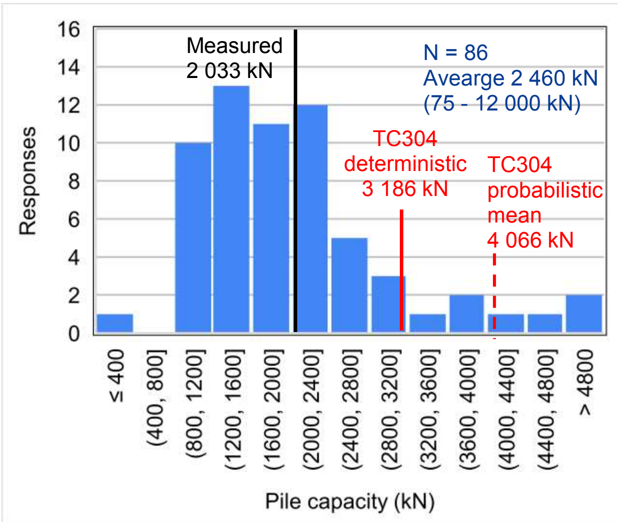


Figure 11. Predicted pile capacity.

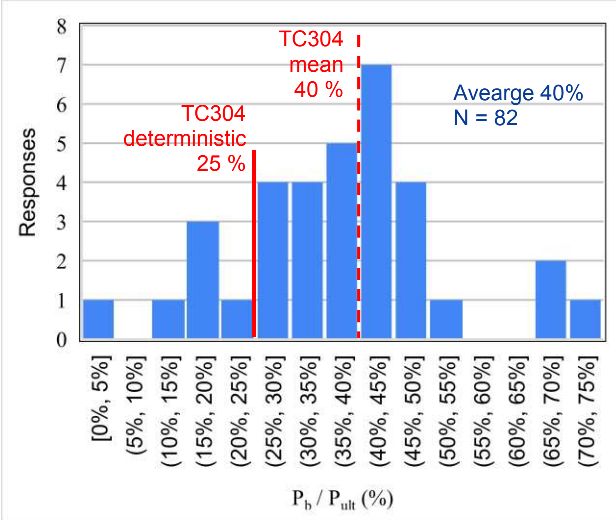


Figure 12. Predicted base capacity as a percentage of total capacity.

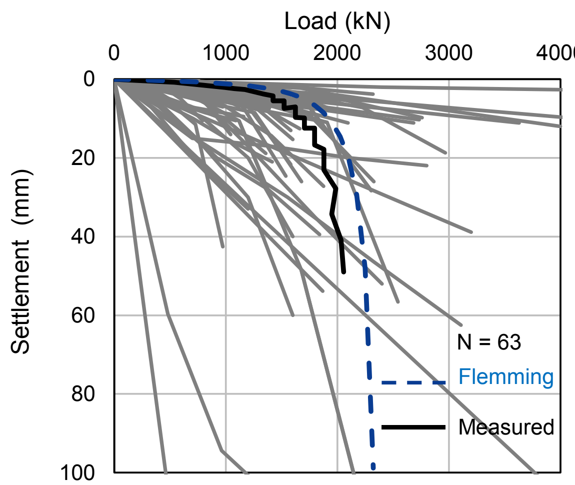


Figure 13. Predicted load settlement curves.

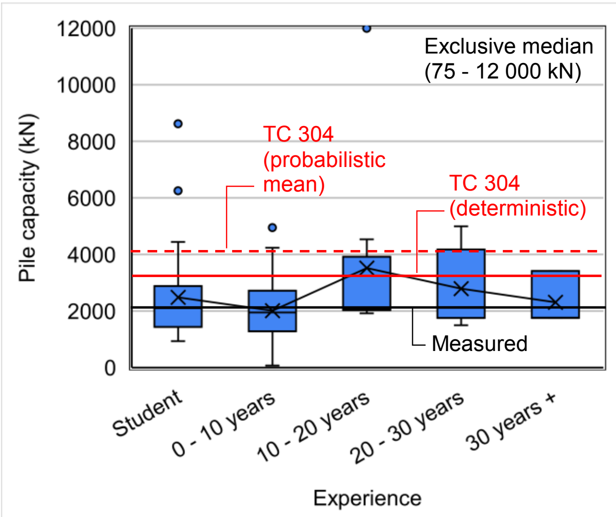


Figure 14. Pile capacity v. experience.

6.3 *Analysis of responses*

Once again, the average values are reasonable but there is a wide range of responses. Figure 14 shows the range of results v. experience of the respondents. This time, the most significant outlier was not from the least experienced respondents. It is clear from this graph how outliers can affect the interpretation of results.

This problem is based on a full-scale pile load test performed at the Texas A&M National Geotechnical Experimentation Site (Briaud 2021a; King et al. 2009), the site from which the test results given in Section 2 were obtained. The result of this pile load test is shown in black solid lines on Figures 11, 13 and 14. The blue dashed line in Figure 13 is a prediction by the first author using the method proposed by Fleming (1992). In this case, the pile capacity was based on empirical correlations with SPT results (Byrne et al. 2019). The pile capacity was assessed as 2 400 kN, the contribution of the base capacity (end bearing) as 25%, and the elastic modulus of the soil below the tip of the pile as 40 MPa. Changing the contribution of the base capacity to 40% (based on Figure 12) gives an even closer alignment between the measured load settlement curve and the results given by Fleming's method.

It is interesting to note that many respondents opted for a straight-line load-settlement response. Some respondents even opted for a curve where the axial stiffness of the pile increased with load. Neither of these options accords with the typical load-settlement response shown by the two curves superimposed on Figure 13.

7 CLAY 3: LATERALLY LOADED PILE

7.1 *The problem (Figure 15)*

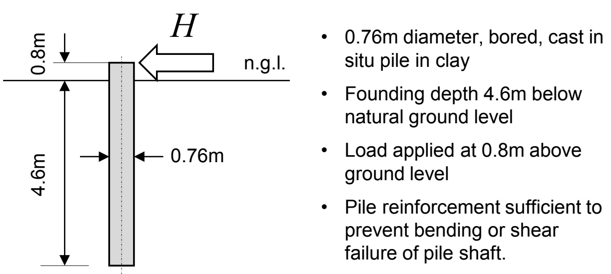


Figure 15. CLAY 3 – laterally loaded pile in clay.

CLAY 3 is a prediction problem in which respondents were asked to predict the ultimate lateral load capacity of the pile, and the load-deflection curve up to 80% of the failure load. Like problem CLAY 2, this problem is based on tests conducted on the Texas A&M National Geotechnical Experimentation Site.

7.2 *Responses*

The predicted lateral load capacity is given in Figure 16 and the load-deflection curves in Figure 17. The TC 304 analysis was undertaken by Shuku and Hachich (2020b).

7.3 *Analysis of responses*

In the Figures 16 and 17, two very optimistic responses were omitted as they were so high that they would have distorted the analysis.

The measured values (black solid lines) in Figures 16 and 17 come from a test on a pile of the same dimensions at the clay site at Texas A&M in 1979 as reported by Bierschwale et al. (1981). It is pleasing to note that the measured lateral load capacity of the pile corresponds well with both the mean and the median values from the responses received.

The analysis by TC 304 (Shuku and Hachich 2020b) was based on an approximation of the shape of the load-deflection curve using a bi-linear function shown by the dashed red line in Figure 17. The load-deflection curve represented by the bi-linear approximation has a shape similar to the measured results in Figure 17.

Figure 18 shows the predicted lateral load capacity of the pile plotted against the undrained shear strength, where this was stated by the respondents. The solid black line in this figure is the lateral resistance from the Broms (1964) method. The deterministic value from the TC 304 calculation is shown by the red cross.

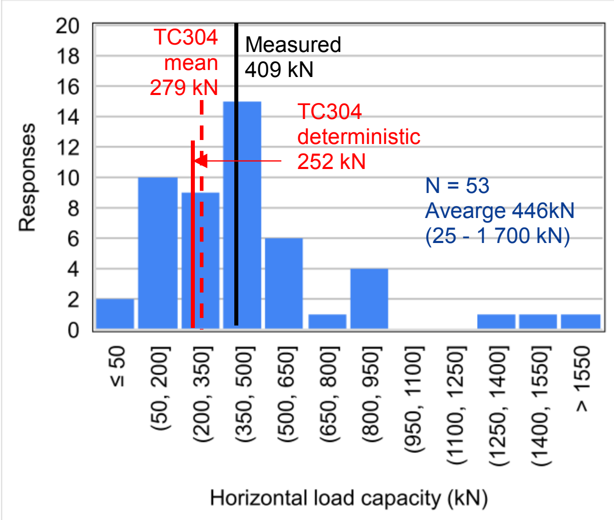


Figure 16. Predicted horizontal load capacity.

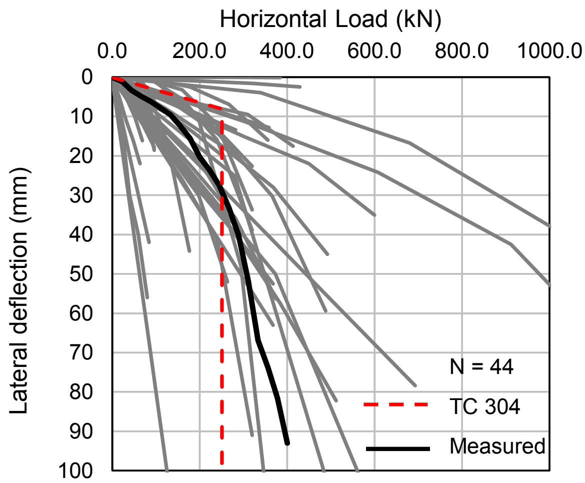


Figure 17. Load-deflection curves.

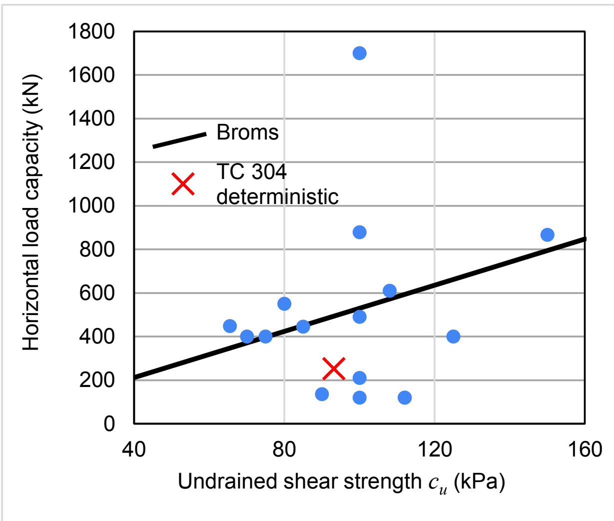


Figure 18. Comparison of results for varying values of shear strength.

Figure 19 shows the range of results v. experience of the respondents.

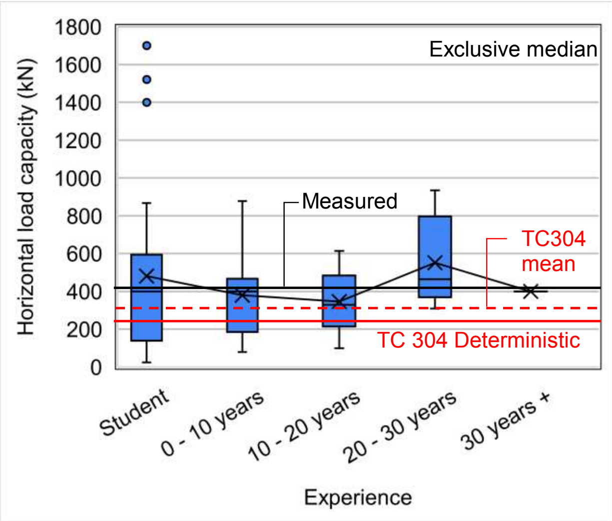


Figure 19. Predicted horizontal load capacity v. experience.

8 CLAY 4: PERMANENT SLOPE

8.1 *The problem (Figure 20)*

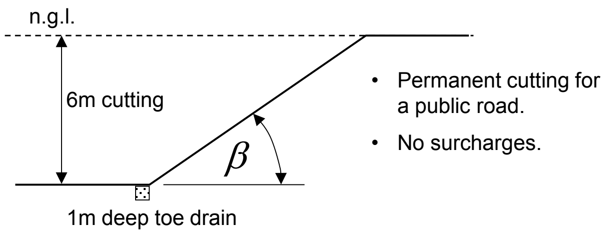


Figure 20. CLAY 4: Permanent slope in clay.

Unlike the previous problems which involved the prediction of the capacity of piles and footings, this is a design problem. There is therefore an added component, namely allowance for a margin of safety. In this case, respondents were asked to specify the maximum slope angle for a permanent road cutting in clay.

8.2 *Responses*

Figure 21 summarises the responses received.

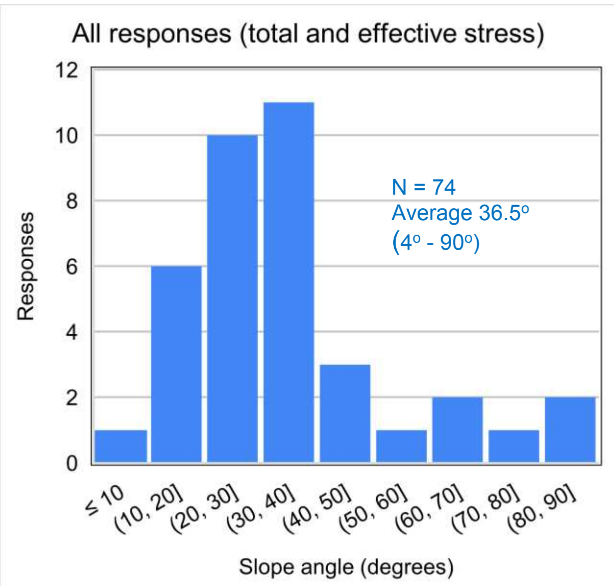


Figure 21. Design angle of cutting (all responses).

When analysing the slope, the respondent must decide whether to use an effective stress analysis (drained analysis, based on c' and ϕ') or a total stress analysis (undrained analysis, based on c_u). Of the 61 respondents who indicated which analysis was used, 36% used an effective stress analysis and 56% a total stress analysis, and 8% used both. The results from the effective and total stress analyses are given in Figures 22 and 23.

The other decision to be made is whether to include a tension crack and whether the tension crack can fill with water. Only 13% of respondents made provision for a tension crack and only 7% allowed the crack to fill with water.

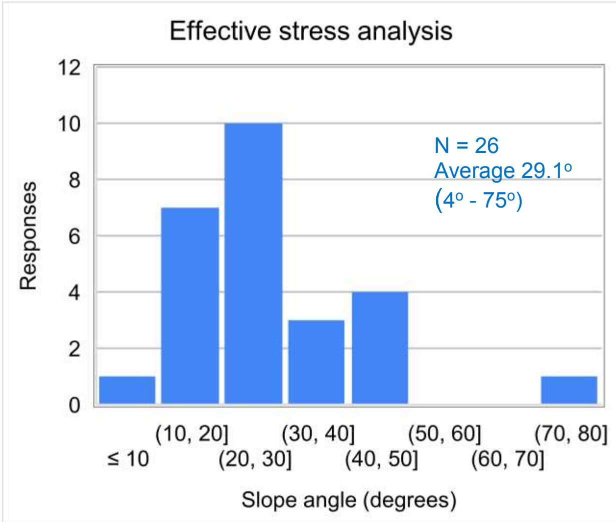


Figure 22. Design angle of cutting (effective stress analysis).

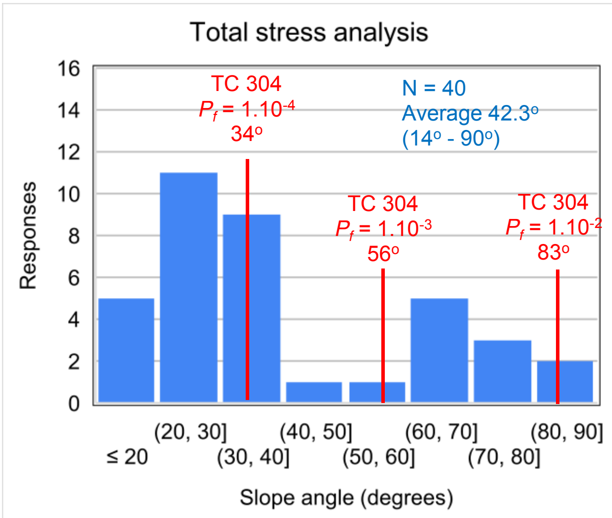


Figure 23. Design angle of cutting (total stress analysis).

8.3 *Analysis of responses*

Given that the clay is overconsolidated and that the slope is to be permanent, there is a strong possibility that failure will occur in the drained state after a prolonged period of time. This seems to be borne out by Figures 22 and 23 which show a lower design angle for the effective stress (drained) analysis than for the total stress (undrained) analysis.

In the undrained condition, it is likely that the slope can be cut vertically. This was confirmed by the analysis by TC 304 (Jiang et al. 2020a) which showed a factor of safety exceeding 2.0 for an undrained, uncracked, vertical cut. However, such a slope is unlikely to remain stable in the long term due to the formation of tension cracks and desiccation of the clays leading to ravelling and sloughing.

The reliability analysis by TC 304 (Jiang et al. 2020b) showed that the probability of failure of an undrained, uncracked slope cut at 83° is 1.10^{-2}, at 56° is 1.10^{-3} and at 34° is 1.10^{-4}.

There are too many different combinations of parameters to permit a more theoretical analysis of this problem.

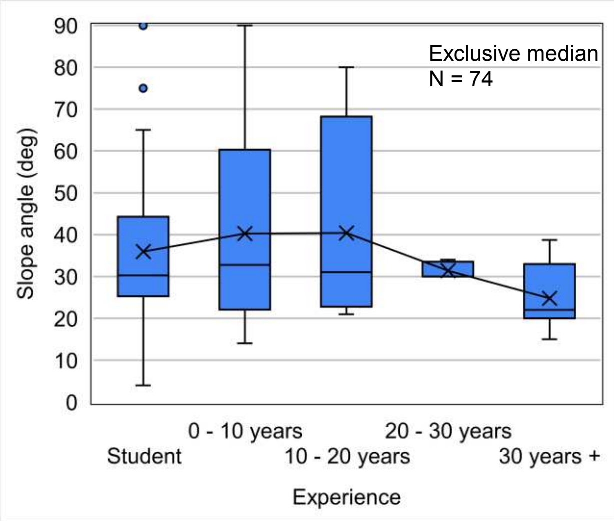


Figure 24. Design angle of cutting v. experience of respondents.

9 SAND 1: VERTICALLY LOADED SPREAD FOOTING

9.1 *The problem (Figure 25)*

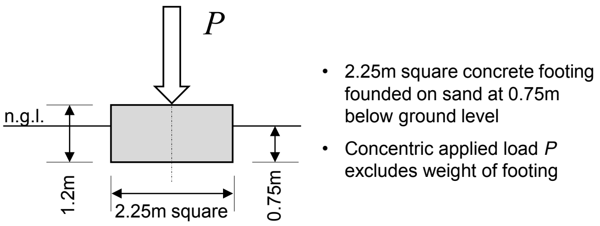


Figure 25. SAND 1: Vertically loaded square footing.

Respondents were asked to predict the maximum load that the footing could carry before failing in bearing and the load that would induce 25mm settlement.

This problem is based on full-scale load tests carried out on five spread footings on sand at the Texas A&M National Geotechnical Experimentation Site in 1993 (Briaud 2021b; Briaud & Gibbens 1997 & 1999). These tests were carried out on square footings with side lengths of 1.0m, 1.5m, 2.5m, and 3.0m (×2).

9.2 *Responses*

The predicted bearing capacity is shown in Figure 26, and the predicted load for 25mm settlement in Figure 27. Figure 28 shows the bearing capacity predictions v. experience.

9.3 *Analysis of responses*

Most respondents assumed a frictional material with no cohesion. Where cohesion was assumed, it was generally small (<5 kPa). Figure 29 shows predicted bearing capacity plotted against the assumed friction angle of the sand. The two curves plotted on this figure are the results of bearing capacity calculations using Meyerhof (1951, 1963 & 1965) and EN1997-1:2004, Annex D. The difference between these curves is due mainly to differences in the shape and depth factors used by the two methods.

TC 304 (Yang & Ching 2020) derived two sets of results, one based on measured values of ϕ and the other with ϕ values derived from SPT and CPT data. These two values are indicated by the two crosses in Figure 29. Both agree reasonably well with the Meyerhof and EC 1997 calculations.

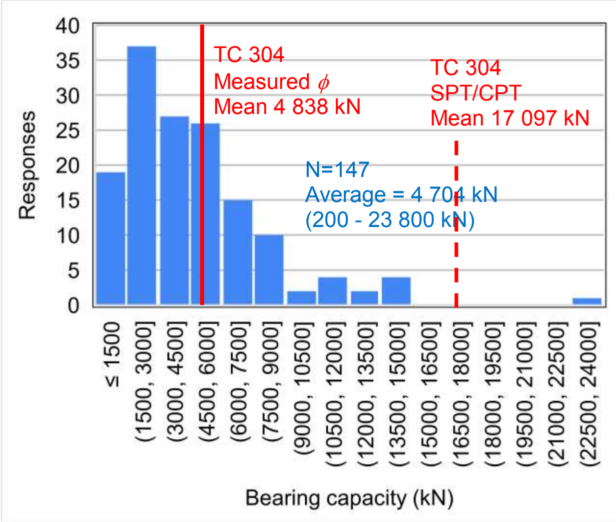


Figure 26. Bearing capacity of footing.

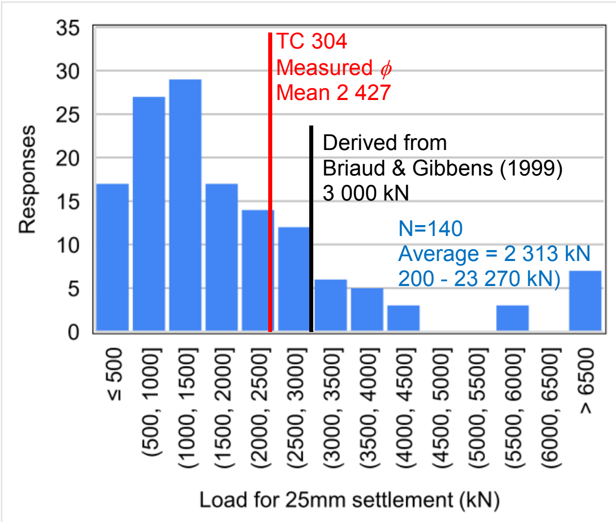


Figure 27. Load for 25 mm settlement.

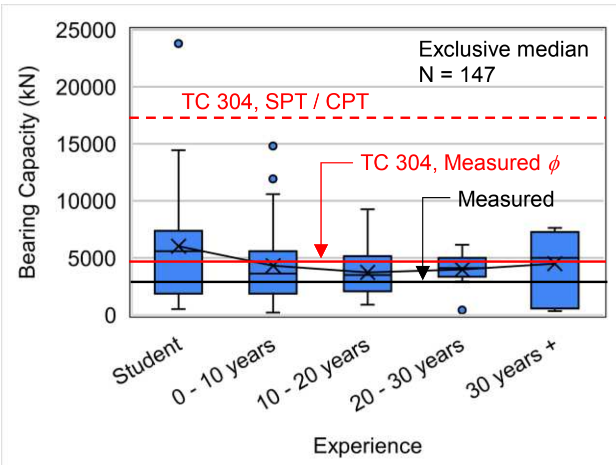


Figure 28. Bearing capacity v. experience.

From Figure 29, most respondents have underestimated the bearing capacity of the footing, some seriously so. The isolated points that plot above the "Meyerhof" line are probably due to the inclusion of cohesion in the calculation. The load that will cause 25mm settlement (P_{25mm}) given in Figure 27 also appears to be underestimated by many respondents. To check whether there is a correlation between low P_{ult} and low P_{25mm} values, the ratio P_{25mm} / P_{ult} was calculated for each response. This ratio varied from 0.001 to 3.15 with an average of 0.59. If the bearing capacity is taken as the maximum load that the footing can carry, values of P_{25mm} / P_{ult} greater than 1.0 do not make sense. Values at the low end of the range should also have raised concern in the mind of the respondent.

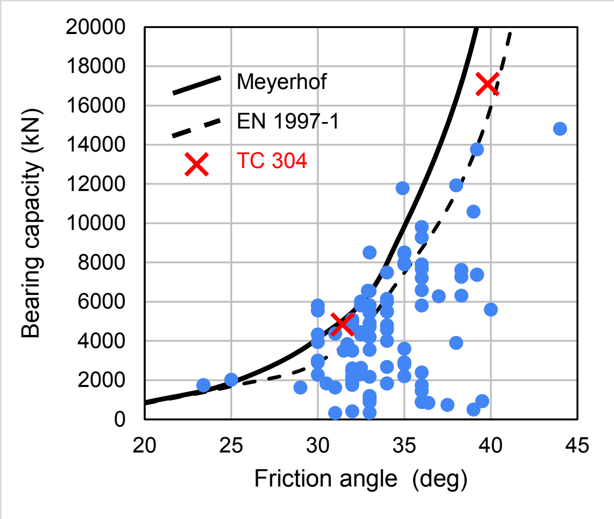


Figure 29. Bearing capacity v. friction angle (ϕ).

Briaud and Gibbens (1999) normalised the load-settlement results from the five full-scale load tests on the square footings referred to in Section 9.1 by dividing the measured settlement by the width of the foundation as shown in Figure 30. All five sets of normalised results plotted in a narrow band. Although the footing size used in the SAND 1 problem (2.25m × 2.25m) differed from the size of the footings tested in 1993, Figure 30 can be used to derive the load-settlement curve for the SAND 1 footing (Figure 31). The bearing pressure causing 25mm settlement (600 kPa) corresponds to an applied load of 3 000 kN which is about 30% higher than the average value from the responses in Figure 26 and the TC 304 value based on measured ϕ values.

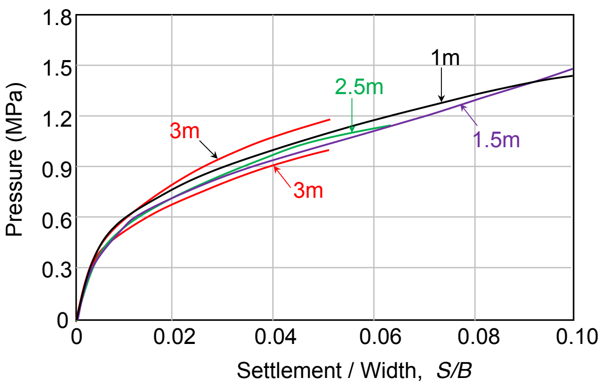
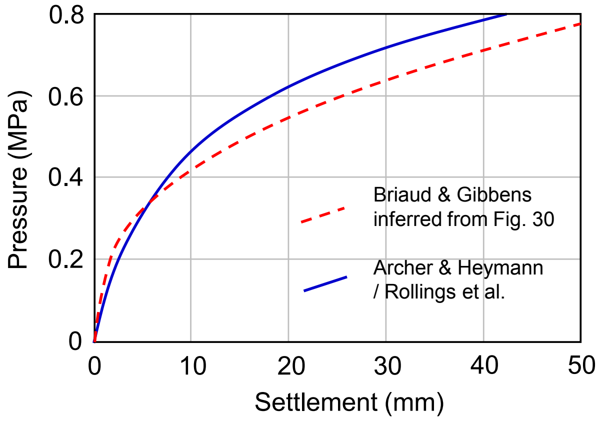


Figure 30. Normalised pressure-settlement for five footings on sand (Briaud & Gibbens 1999).

When performing a routine settlement calculation, such as for the footing in question, many geotechnical engineers would select an appropriate elastic modulus based on the available information and perform a simple elastic settlement calculation. Most of these solutions give a linear relationship between load and settlement which, as illustrated by Figure 30, is clearly not the case for the entire curve, but reasonable up to 50% of the ultimate load. For more complex problems, finite element analysis may be carried out incorporating a constitutive model for which the necessary parameters can be determined or estimated. However, for this problem (SAND 1) and the data available, a simple calculation based on small-strain shear stiffness (G_0) provided the closest approximation to the test results shown in Figure 30.

The small-strain shear stiffness (G_0) data gives an average value of $G_0 \approx 100$ MPa dropping to ≈ 60 MPa between 7m and 9m. This small-strain stiffness profile was analysed using the method of Archer and Heymann (2015) and the stiffness degradation curve proposed by Rollins et al. (1998). The result is shown by the solid blue line in Figure 31. It compares well with the mean of results of the full-scale load tests in Figure 30 for a footing width of 2.25m.

Figure 31. Settlement calculated from G_0 values using Archer and Heymann (2015) and Rollings et al. (1996).



10 SAND 2: AXIALLY LOADED PILE

10.1 *The problem (Figure 32)*

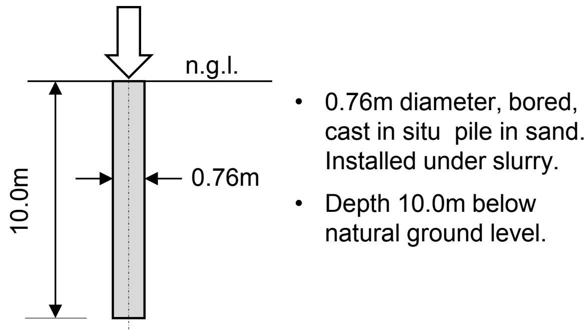


Figure 32. SAND 2: Axially loaded pile.

Respondents were asked to predict the ultimate load capacity of the pile, the base and shaft resistance, and the load-settlement curve up to 80% of the pile capacity.

This problem is based on full-scale tests carried out on drilled shafts in sand at the Texas A&M National Geotechnical Experimentation Site in 2006 (King et al. 2009; Briaud 2021b).

10.2 *Responses*

The pile capacity, base capacity and load-settlement curves are shown in Figures 33, 34 and 35. The pile capacity responses for the various experience categories are given in Figure 36.

10.3 *Analysis of responses*

The average pile capacity predicted by the respondents (3 039 kN) agreed well with the measured pile capacity from the Texas A&M National Geotechnical Experimentation Site (2 937 kN). However, the range of predicted capacities is extremely high, anywhere from 0.14 to 2.9 times the measured value. Forty six out of 82 respondents underpredicted the pile load capacity. It is interesting to note that the group that predicted the highest pile capacity on average was the group with the most experience. The average prediction for this group was over 40% too high.

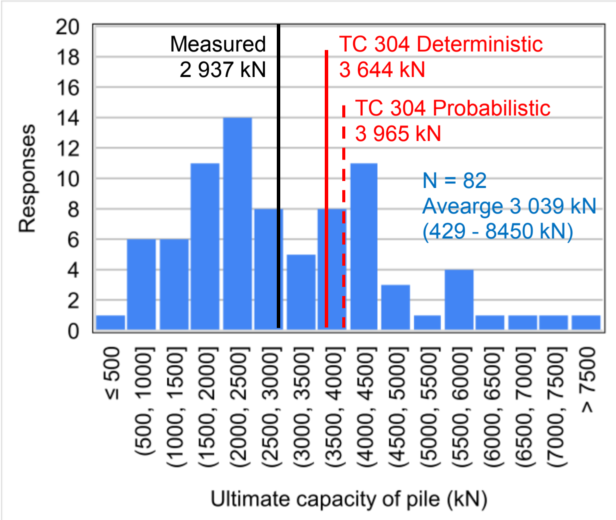


Figure 33. Predicted ultimate capacity of pile.

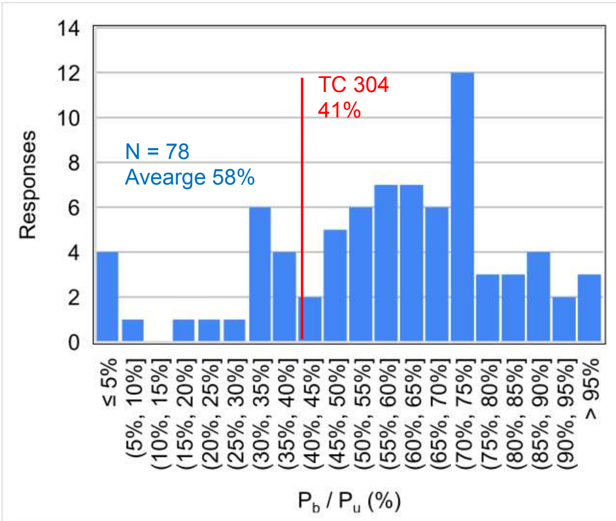


Figure 34. Ratio of base capacity to pile capacity.

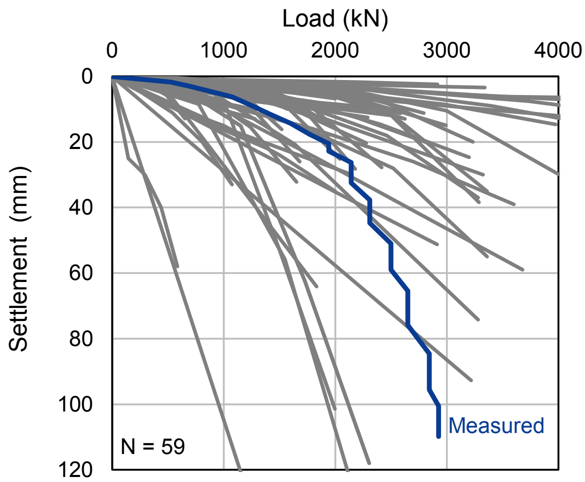


Figure 35. Load-settlement predictions.

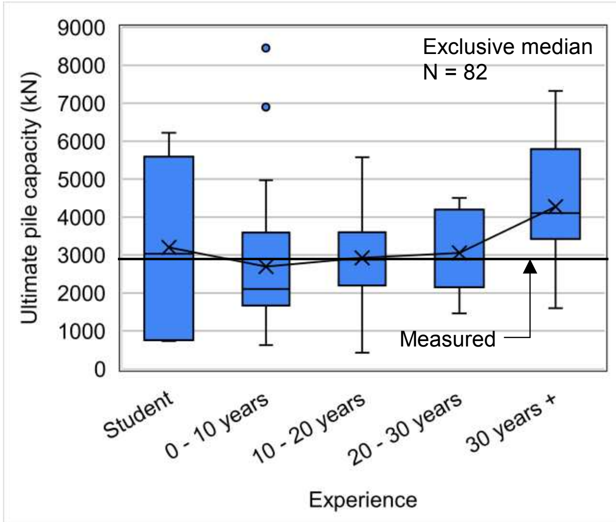


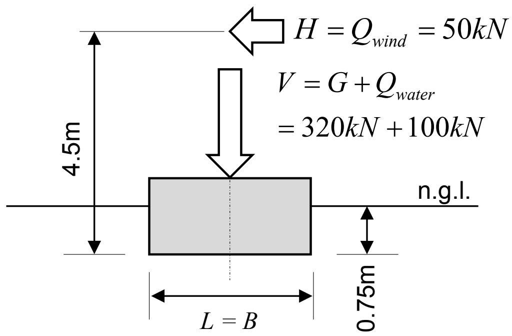
Figure 36. Predicted pile capacity v. experience.

The deterministic and probabilistic predictions by TC 304 (Shuku & Hachich 2020c) in Figure 33 were respectively 24% and 35% higher than the measured resistance.

The load-settlement curves also showed significant variation, with many respondents predicting a linear load-settlement response.

11 SAND 3: SPREAD FOOTING WITH HORIZONTAL LOAD

11.1 *The problem (Figure 37)*



- Foundation is for a water tank stand
- Vertical load V includes dead load G excluding weight of footing) and weight of water in tank Q_{water}.
- Horizontal load H is due to wind only Q_{wind}. Can act in any horizontal direction.
- Loads are characteristic values.

Figure 37. SAND 3: Square footing with horizontal load.

This is a design problem in which the respondents were asked to determine the required size of a base for a water tank stand.

11.2 *Responses*

Ninety-nine responses were received. The size of the footing required by the respondents is given in Figure 38, the variation in required size with friction angle in Figure 39 and the experience of the respondents in Figure 40.

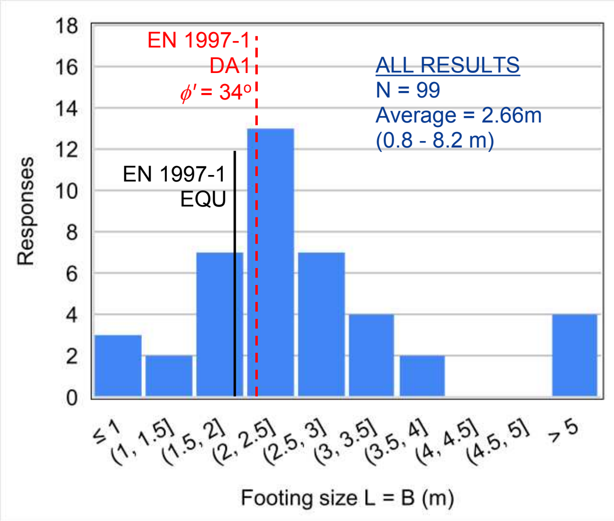


Figure 38. Required footing size (all results – tank full and tank empty).

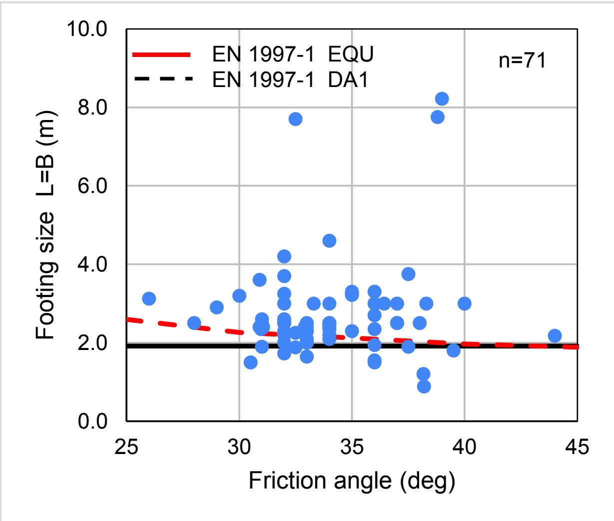


Figure 39. Variation in required size with friction angle.

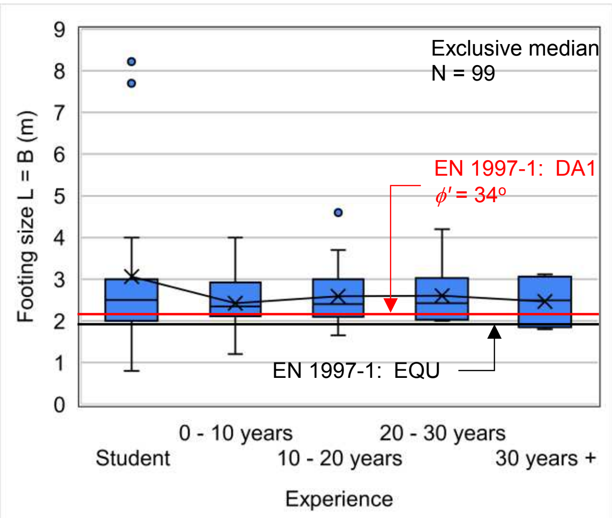


Figure 40. Required footing size v. experience.

From the 71 respondents who stated the friction angle assumed in their calculations, the average friction angle was 34°, varying from 26° to 44°. Unfortunately, it is not possible to determine from the responses whether these are average values or characteristic values.

Forty-two respondents determined the footing size for the "tank empty" condition and 49 for the "tank full" condition.

11.3 *Analysis of responses*

TC 304 did not submit an analysis of this problem, as current knowledge about the model uncertainty for the design method of spread footings with horizontal load is limited (Ching & Zhang 2020). Furthermore, there is no corresponding load test. The responses received were therefore compared to an analysis performed using Design Approach 1 from EN1997-1:2004, together with the default values of partial load and material factors and load combination factors from EN1990:2002 and EN1997-1:2004, following the model solution for a similar problem by Orr (2005). This comparison is shown in Figure 39.

The above analysis shows that the STR (DA1 C1) and GEO (DA1 C2) design approaches yield very similar results. For all but the lowest friction angle considered (25°), the tank empty condition dictates the required size of the base. The relatively small variation in base size with friction angle is due to the dominating effect of eccentricity of load on the calculation. The minimum base size required to satisfy the EQU limit state is 1.91m. If unfactored values are used, the base will overturn if the base is smaller than 1.24m.

Based on Figure 39, most of the respondents overestimated the required size of the base.

12 SAND 4: PERMANENT ROAD CUTTING

12.1 *The problem (Figure 41)*

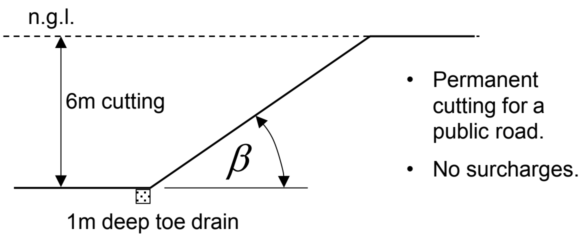


Figure 41. SAND 4: Permanent road cutting.

This is a design problem in which respondents were requested to provide the angle at which the slope should be constructed.

12.2 *Responses*

Sixty-nine responses were received. The required slope angles are shown in Figure 42, the slope angle required for various friction angles in Figure 43 and the slope angle versus experience of the respondents in Figure 44.

Where working load design methods were used, the average value of the factor of safety used by the respondents was 1.5, ranging from 1.2 to 2.0.

The average friction angle was 33.8° varying from 25° to 41°.

12.3 *Analysis of responses*

The solid red line in Figure 43 is from a Morgenstern-Price slope stability analysis (Morgenstern & Price 1965) performed using commercial software (Geostudio SLOPE/W, version 2021 R2) with a factor of safety of 1.5. This line is virtually indistinguishable from the result of an infinite slope analysis with a factor of safety of 1.5, i.e. $\alpha = arctan\ (tan\phi/1.5)$.

For the average friction angle used by the respondents of 33.8°, the average slope angle of 26° (from Figure 42) corresponds to a factor of safety of 1.37. This is a reasonable value for a slope where the failure surface is likely to be shallow and the quantity of material involved in the failure will be small.

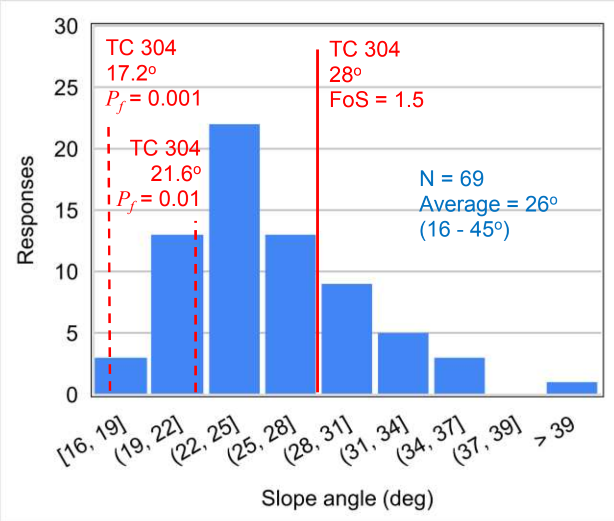


Figure 42. Required slope angle.

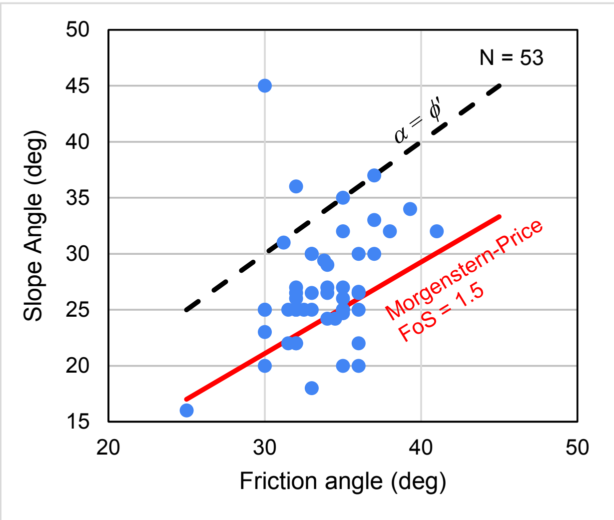


Figure 43. Variation in required slope angle with friction angle.

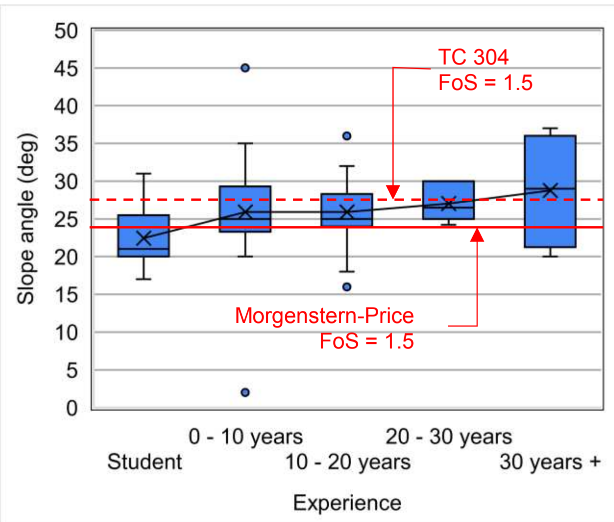


Figure 44. Experience of designers.

It is interesting to note that three respondents assumed a slope angle equal to the friction angle (i.e. $\alpha = \phi$). This implies a factor of safety of 1.0. Two respondents selected slope angles above the friction angle. For such slopes to be stable, some cohesion must be present.

Although a slope angle of 16° (minimum submitted value) may seem low, the analyses by TC 304 (Jiang et al. 2020b) found that a slope angle of 17.2° resulted in a probability of failure of 1.10^{-3} and that the slope would have to be flattened to less than 14° to achieve a failure probability of 1.10^{-4}.

13 SAND 5: PROPPED EMBEDDED RETAINING WALL

13.1 *The problem (Figure 45)*

This is a design problem in which respondents were requested to determine the required depth of embedment (d), the propping force (P) and the maximum bending moment in the wall (M_{max}).

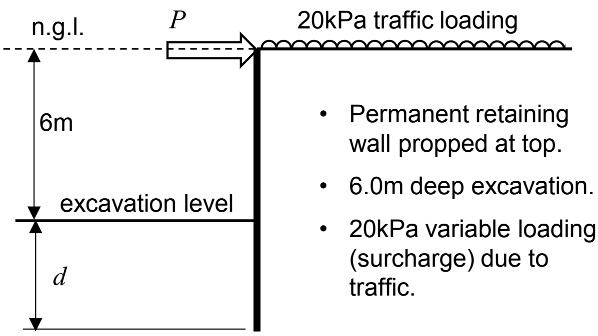


Figure 45. SAND 5: Propped embedded retaining wall.

13.2 *Responses*

Based on the responses received, the required embedment depth is shown in Figure 46, the propping force in Figure 47 and the minimum required bending resistance of the wall in Figure 48.

The average friction angle used by the respondents in their calculations was 34°. Again, there is insufficient information to ascertain if this is an average value or a characteristic value.

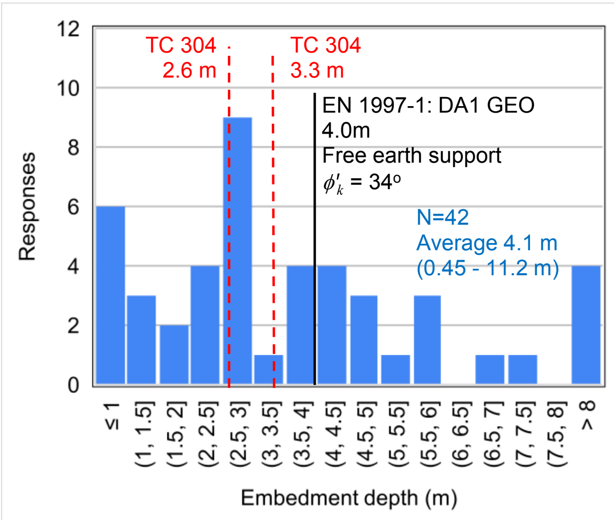


Figure 46. Required embedment depth.

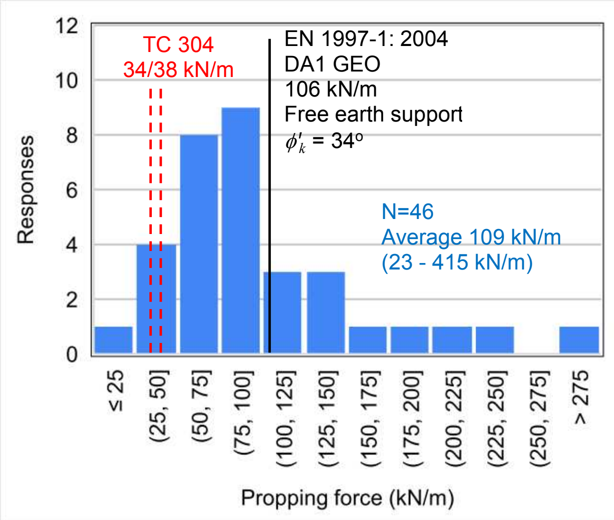


Figure 47. Required propping force.

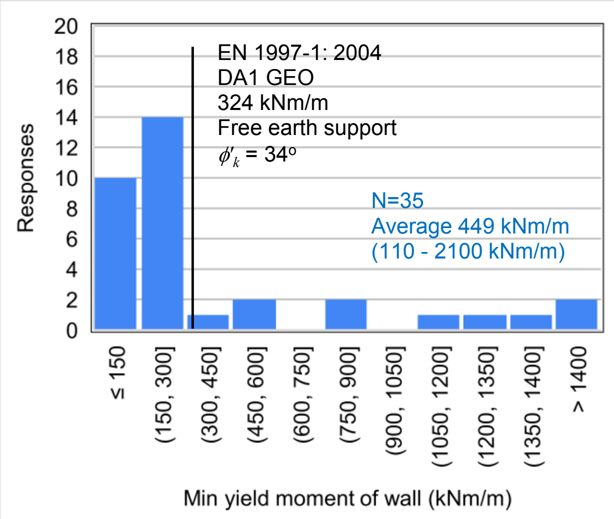


Figure 48. Required minimum moment capacity of wall.

13.3 *Analysis of responses*

No full-scale load test on a similar structure has been carried out.

TC 304 (Shuku et al. 2020) performed a deterministic and probabilistic analysis of the depth of embedment and the propping force required. Their responses are shown by the dashed red lines in Figures 46 and 47.

For a further comparison, a limit states design was carried out based on Design Approach 1, Combination 2 (GEO) from EN1997-1:2004. The average characteristic value of the friction angle (ϕ'_k) was taken as 34°, equal to the average friction angle used by the respondents. This calculation, which made use of the free-earth support method, was based on the model solution for a propped cantilever wall prepared by Orr (2005). The results are shown by the solid black lines in Figures 46 to 48.

There is insufficient data to allow meaningful comparison of the responses in terms of experience.

14 SAND 6: SOIL NAILED WALL

14.1 *The problem (Figure 49)*

This is a design problem in which respondents were requested to determine the required length of soil nails (L), and the maximum horizontal spacing (S_H).

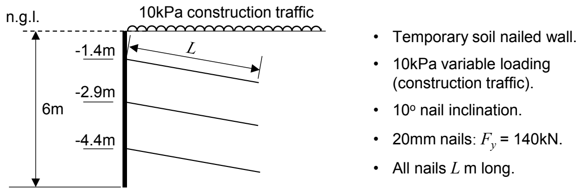


Figure 49. SAND 5: Propped embedded retaining wall.

14.2 *Responses*

From the replies of 38 respondents, the required nail length, the horizontal spacing of the nails and the total length of nails per metre length of face are given in Figures 50 to 52.

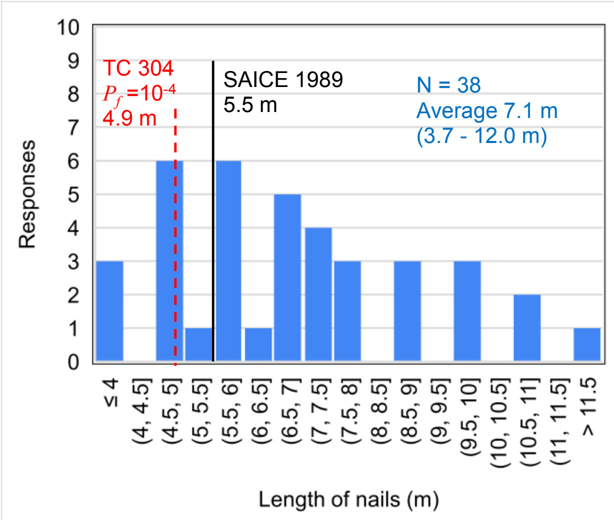


Figure 50. Required length of nails.

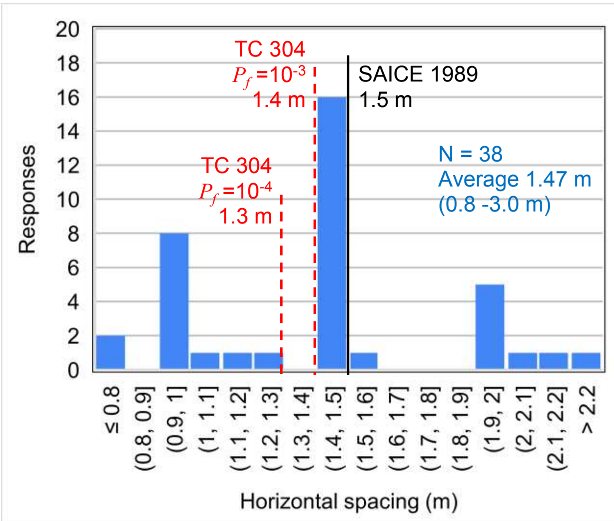


Figure 51. Horizontal spacing of nails in each row.

14.3 *Analysis of responses*

The construction of soil-nailed walls is a highly competitive market in South Africa and full advantage is taken of the typically partially saturated, relatively stiff soil profile. Design is carried out in accordance with a local lateral support code (SAICE 1989) using a single- or double-wedge analysis with a FoS of 1.5 on wedge stability and a FoS of 2.0 on the bond stress. This method of analysis has been used for comparison purposes for this problem as shown by the solid black lines in Figures 50 to 52. The analysis again used the average friction angle of 34° obtained from the responses received. In terms of total meterage of soil nails, the results compare favourably with those from the analyses by TC 304 (Cao et al. 2020).

As correctly pointed out by TC 304, the nail length and the nail spacing are not independent. Wider spacings require longer nails. However, the nail spacing and length of nails can be combined into a single variable for comparison purposes, namely the total length of nails per metre of face as shown in Figure 52. The average length of nails per metre from the survey (15.9m) is greater than required.

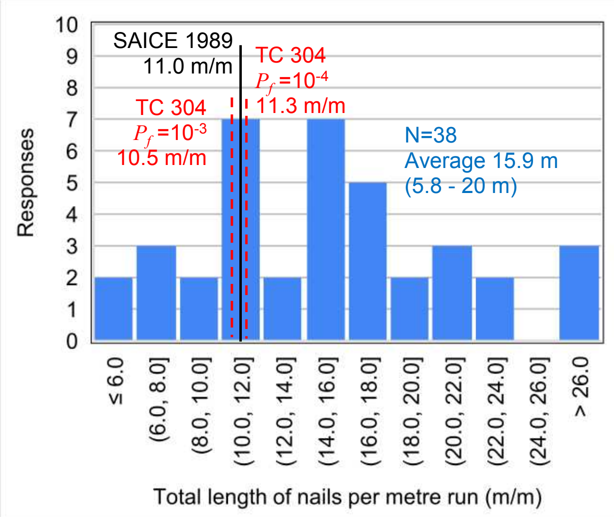


Figure 52. Total length of nails per metre length of face.

For this specific problem, an increase in nail spacing results in an increase in the length of nails but an overall decrease in the length of nails per metre length of face. However, there is a practical maximum spacing of nails, typically 1.5m in sandy soils such as present on the sand site. Spacings of 3m (maximum from responses received) may result in less total meterage of nails but is likely to give problems during construction. The average spacing of 1.47m is closer to the norm.

The rule of thumb for the length of soil nails is 0.7× – 0.9× the height of the face. Designs using shorter soil nails are often governed by limiting bond stresses, while designs with longer nails may be governed by the yield strength of the nail. In the SAICE and TC 304 designs, the bar lengths are 0.92 and 0.82 times the height of the face respectively. The average length of nails from the responses received (7.1m) is higher than is normally expected.

15 DISCUSSION

15.1 *Correctness and adequacy of geotechnical data*

Some respondents questioned the adequacy and correctness of the data. In particular, the CPT data was subject to much scrutiny, and inconsistencies in the data were pointed out.

It should be borne in mind that some of the data comes from the 1980s (Briaud 1997) when testing methods were less advanced than they are today, and digital recording of data was non-existent. As a result, the data was digitised from the plotted soil test results extracted from published papers, as the original data was not available in modern electronic form.

The compilers of the survey acknowledge discrepancies in the data. However, geotechnical engineering is not an exact science. In real life, geotechnical designers seldom have the quantity and quality of data they would like, and they have to make do with what is available. This survey is thus a better reflection of conditions in geotechnical engineering practice than it would have been with a well-manicured set of test results.

Having said that, it is worth reflecting on the effect on the outcome of the survey of both the data itself and interpretation of the data. As stated in the introduction, the survey concerned two types of problems – prediction and design. The prediction problems were aimed at assessing the selection of parameters and calculation models. The design problems were aimed additionally at assessing the provision made for safety and serviceability in geotechnical design. There are thus three things that are being assessed: (1) the selection of design parameters from test results, (2) the methods of analysis (calculation models) used, and (3) the provision made for safety/serviceability. The results of the survey would have been easier to analyse had the survey been split into three sections, dealing with each of the above issues. However, that would have reduced the survey to the level of an undergraduate assignment and would not have assessed the full range of design solutions put forward by the respondents to this survey.

15.2 *Assessment of responses*

Probably the most surprising result of this survey is the wide range of responses to almost every problem. The obvious starting point for any analysis would be to compare the predictions and the designs to known outcomes based on either measurement or the application of established theory. However, as stated above, geotechnical engineering is not an exact science. Even where we have measured values from full-scale tests on the sites from which the data originated, these are typically single test results which may either be below or above the mean of a larger number of test results. Nevertheless, a best estimate of the result is a better starting point than no estimate at all.

To this end, the measured values from full-scale tests on the National Geotechnical Experimentation Site have been adopted as best estimates. Where no test results are available, the results from calculations based on widely accepted codes or calculation models have been used. The input values to these calculations have been taken as the mean value of the parameters given in the responses received. The authors do not contend that the best-estimate values are "correct". The complexity of geotechnical design and the uncertainties involved make it impossible to say what is correct or incorrect. These values are simply a starting point for the rational assessment of the results.

Table 1 summarises the responses received, grouping them into predicted values (load capacity, load for 25mm settlement, etc.), design dimensions (size of footing, slope angle, etc.) and parameter values (cohesion and friction angle). In the final three columns of Table 1, the range (minimum and maximum) and mean (average) values from the responses have been normalised by dividing by the best-estimate value.

Table 1. DEM input parameters

Result		Best Estimate			Responses			Normalised		
Problem	Parameter	Estimate		Source	Min	Mean	Max	Min	Mean	Max
Prediction										
CLAY 1	P_{ult}	1154	kN/m	Mean TC 304	43	1127	8872	0.04	0.98	7.69
CLAY 1	P_{25}	650	kN/m	TC 304	18	538	2413	0.03	0.83	3.71
CLAY 2	P_{ult}	2033	kN	Measured	75	2460	12000	0.04	1.21	5.90
CLAY 3	P_{ult}	409	kN	Measured	25	446	1700	0.06	1.09	4.16
SAND 1	P_{ult}	6235	kN	EN1997 $\phi' = 34^{\circ}$	200	4704	23800	0.03	0.75	3.82
SAND 1	P_{25}	2900	kN	Measured	200	2313	23270	0.07	0.80	8.02
SAND 2	P_{ult}	2937	kN	Measured	429	3039	8450	0.15	1.03	2.88
Design dimension/slope										
CLAY 4	α undrained	34	deg	TC 304 $P_f = 10^{-4}$	14	40.9	90	-	1.20	-
SAND 3	L = B	2.07	m	EN1997 $\phi' = 34^{\circ}$	0.8	2.66	8.2	0.39	1.29	3.96
SAND 4	α	24	deg	Inf. slope $\phi' = 34^{\circ}$	16	26	45	-	1.08	-
SAND 5	d	4	m	EN1997 $\phi' = 34^{\circ}$	0.45	4.1	11.2	0.11	1.03	2.80
SAND 5	P	106	kN/m	EN1997 $\phi' = 34^{\circ}$	23	109	415	0.22	1.03	3.92
SAND 5	M_{req}	324	kNm/m	EN1997 $\phi' = 34^{\circ}$	110	449	2100	0.34	1.39	6.48
SAND 6	m/m	11	m/m	*SAICE* $\phi' = 34^{\circ}$	5.8	15.9	20	0.53	1.45	1.82
Material property										
CLAY 1	c_u	111	kPa	TC 304 CLAY 1&4	51	98	199.3	0.46	0.88	1.80
CLAY 3	c_u	111	kPa	TC 304 CLAY 1&4	65	98	150	0.59	0.88	1.35
CLAY 4	c_u	111	kPa	TC 304 CLAY 1&4	41	77	150	0.37	0.69	1.35
SAND 1	ϕ'	35.6	deg	TC 304 mean	23.4	33.8	44	0.66	0.95	1.24
SAND 3	ϕ'	35.6	deg	TC 304 mean	26	34	41	0.73	0.96	1.15
SAND 4	ϕ'	35.6	deg	TC 304 mean	25	33.8	41	0.70	0.95	1.15

The most significant outcome is that the average value of the responses typically falls within a range of 30% above or below the best estimate value, i.e. the normalised mean in the second last column typically lies between 0.7 and 1.3.

As far as the range of responses is concerned, the smallest range was in the material properties assessed from the available geotechnical data (c_u for clay and ϕ' for sand). The likely reason for this small range is that most experienced respondents will have a feel for the values of these parameters. Some results are regarded as highly optimistic or pessimistic, but very few are highly improbable. It is noted that the material properties submitted by the respondents include both best-estimate values and characteristic values. In the context of the problems included in this survey, characteristic values are typically lower than best-estimate values. This will tend to skew the average and the range of the responses towards the lower end of the scale. This is a shortcoming of the survey, but one which could be addressed by further analysis of the results.

The next lowest range was for design dimensions. Once again, most experienced respondents will have a feel for the correctness of a result. Nevertheless, the design dimensions submitted include values that are highly improbable, being either unsafe or excessively conservative.

The highest range of responses was for the prediction problems, all of which relate to load capacity. This is somewhat surprising as prediction of load capacity includes only two main categories of uncertainty, namely parameter uncertainty and calculation model uncertainty, while the assessment of design dimensions includes a third category of uncertainty, namely provision for safety. Despite the average values of the predictions being reasonable estimates, the extremes in virtually all cases are highly improbable.

16 CONCLUSIONS

As a result of the very wide range of responses to almost every problem in the survey, it is difficult to provide a clear answer to the question, "Are we overdesigning?" The only problem where there appeared to be a clear case of overdesign (underprediction of capacity) was the prediction of the failure load for problem SAND 1 (vertically loaded spread footing). This is surprising, as the calculations involved are straight-forward and the calculation models are well established. There was also a tendency towards overdesign of the base size required for SAND 3 (footing on sand with horizontal load). This is more understandable, as the problem is more complex and the methods of calculation less clearly defined. In all other cases there was evidence of both overdesign and underdesign.

A point of concern is the apparent failure, even among experienced designers, to consider potentially critical load cases or recognise clearly incorrect predictions. Examples are the failure to consider the possibility of drained slope failure in over-consolidated clays (CLAY 4, permanent slope cut), critical load conditions (e.g. tank empty in problem SAND 3) and critical limit states (e.g. the EQU limit state in SAND 3).

In all cases where measured values are available, accepted methods of analysis yielded similar values. The wide range of responses received points to two possibilities. The first is that established methods of analysis are not being applied or are being incorrectly applied. The second is that less attention was paid to the analysis of the survey problems than would be paid to geotechnical design in practice, with less detailed checking of calculations, and less critical scrutiny of the final result. It is concerning that simple checks could have ruled out many of the improbable responses.

Much more can be done with the data collected during this survey. This is why this article was written and why the data is available on the websites cited in the article.

The results of this survey clearly indicate that the ISSMGE should pay as much attention to the application of existing knowledge as to the creation of new knowledge. More of our technical activities should involve practitioners and be aimed at improving the state of practice within the industry.

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ANNEXURE A: CLAY SITE PROPERTIES

A1 *Laboratory tests*

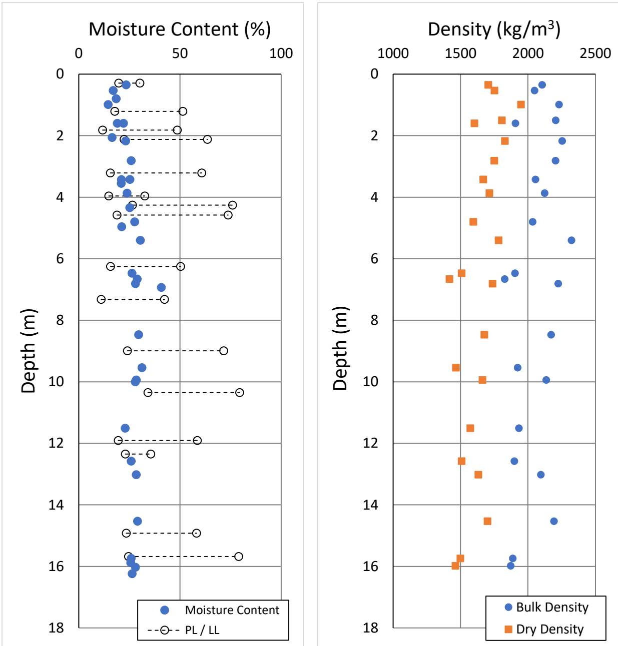

Figure A1. CLAY: moisture content and density.

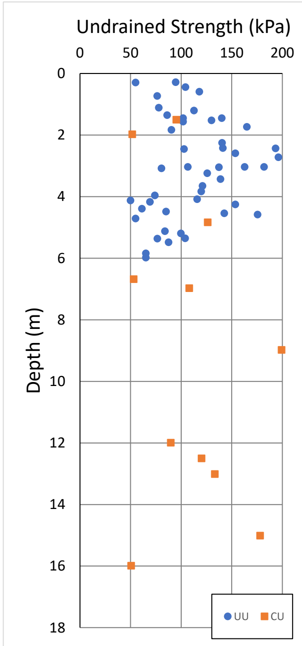

Figure A3. CLAY: undrained shear strength.

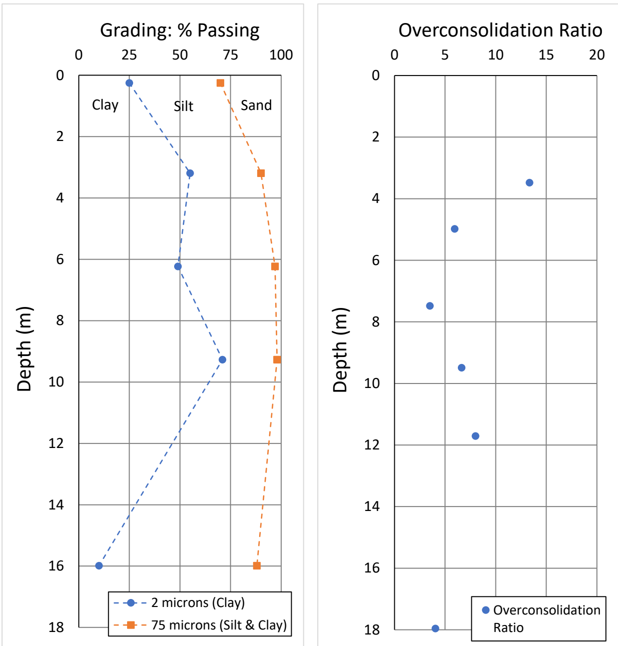

Figure A2. CLAY: grading and over-consolidation ratio.

A2 *Field tests*

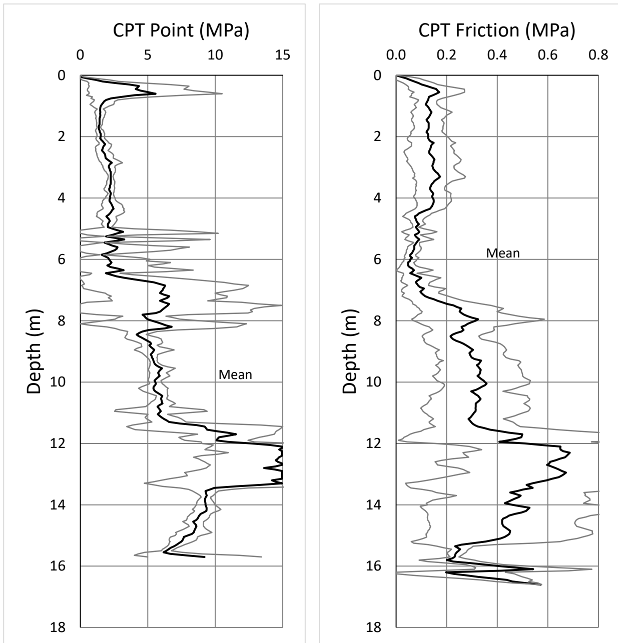

Figure A4. CLAY: CPT point and friction resistances.

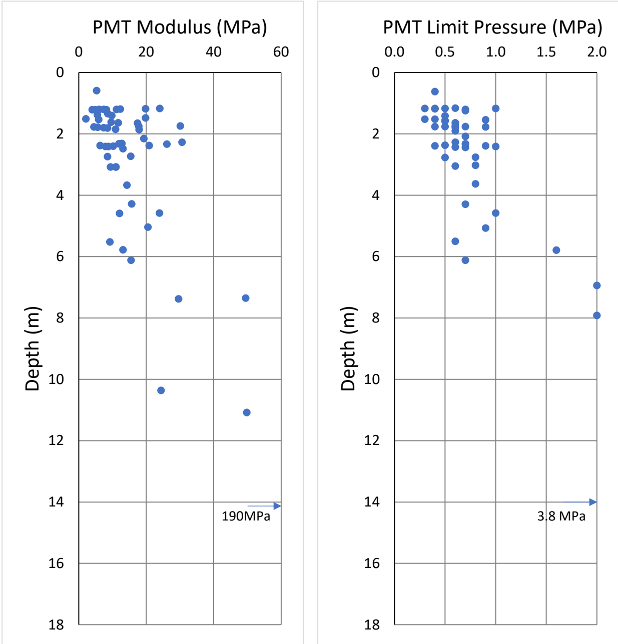

Figure A5. CLAY: Pressuremeter modulus and limit pressure.

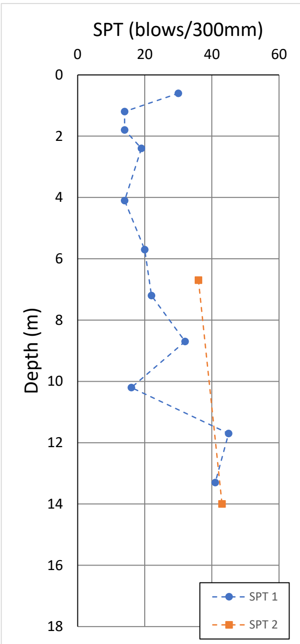

Figure A6. CLAY: SPT tests

ANNEXURE B: SAND SITE PROPERTIES

B1 *Laboratory tests*

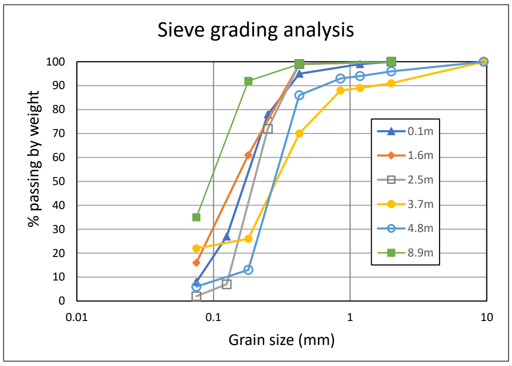

Figure B1. SAND: grading analysis.

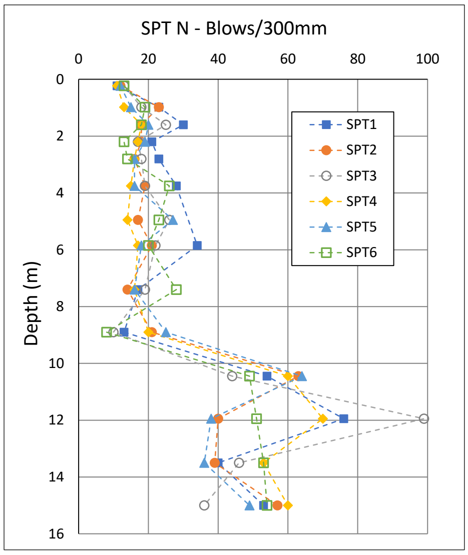

Figure B2. SAND: SPT tests.

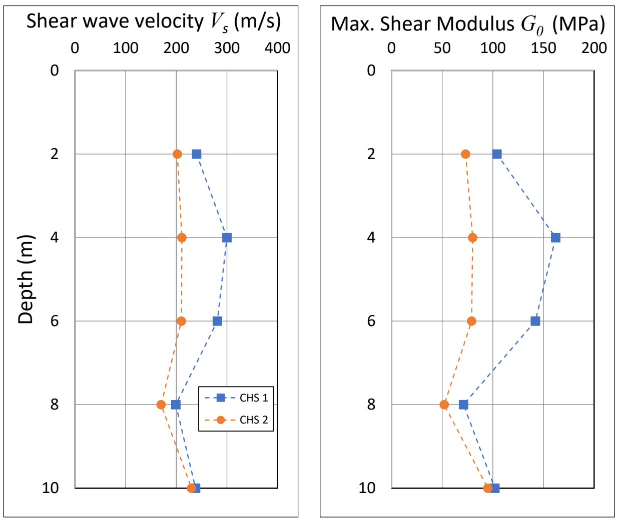

Figure B3. cross-hole seismic tests.

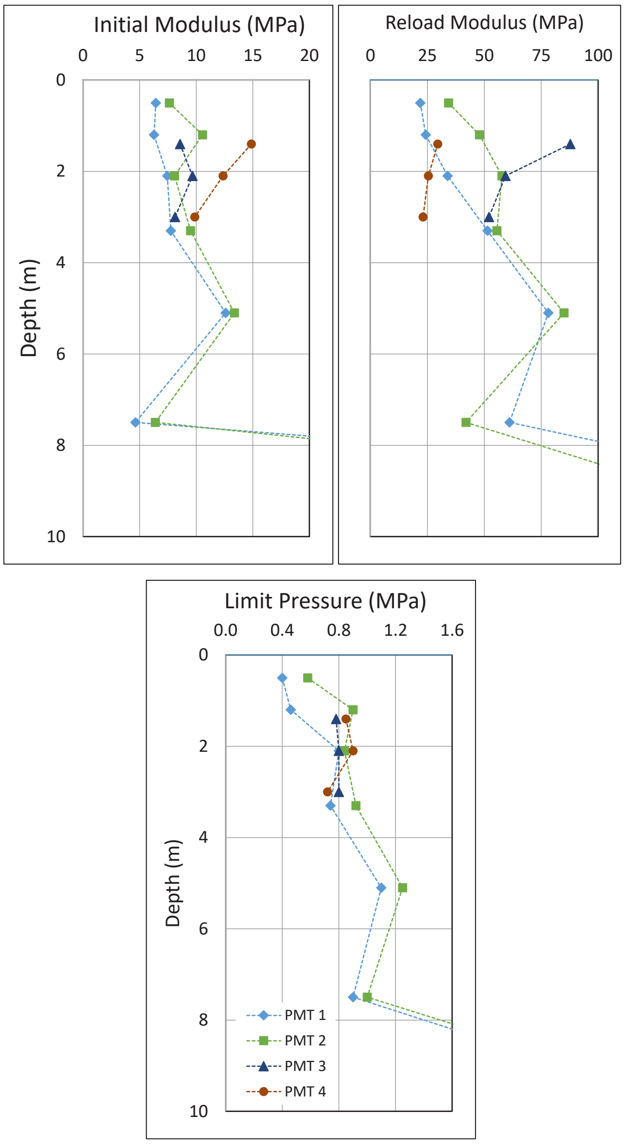

Figure B4. SAND: pressuremeter tests.

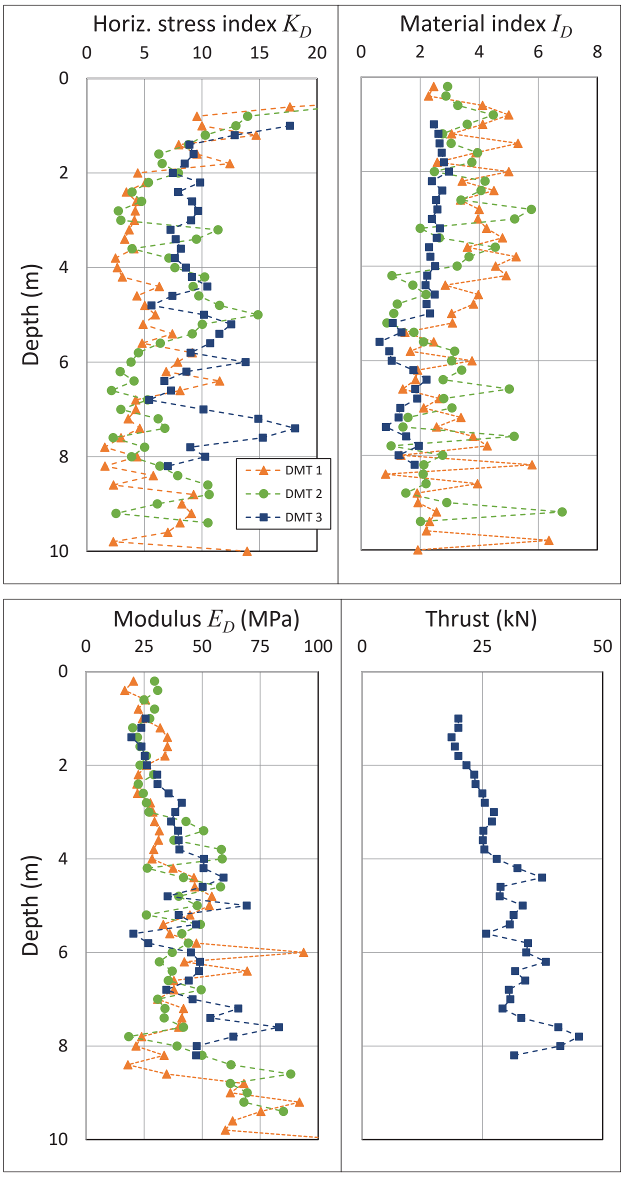

Figure B5. SAND: dilatometer tests.

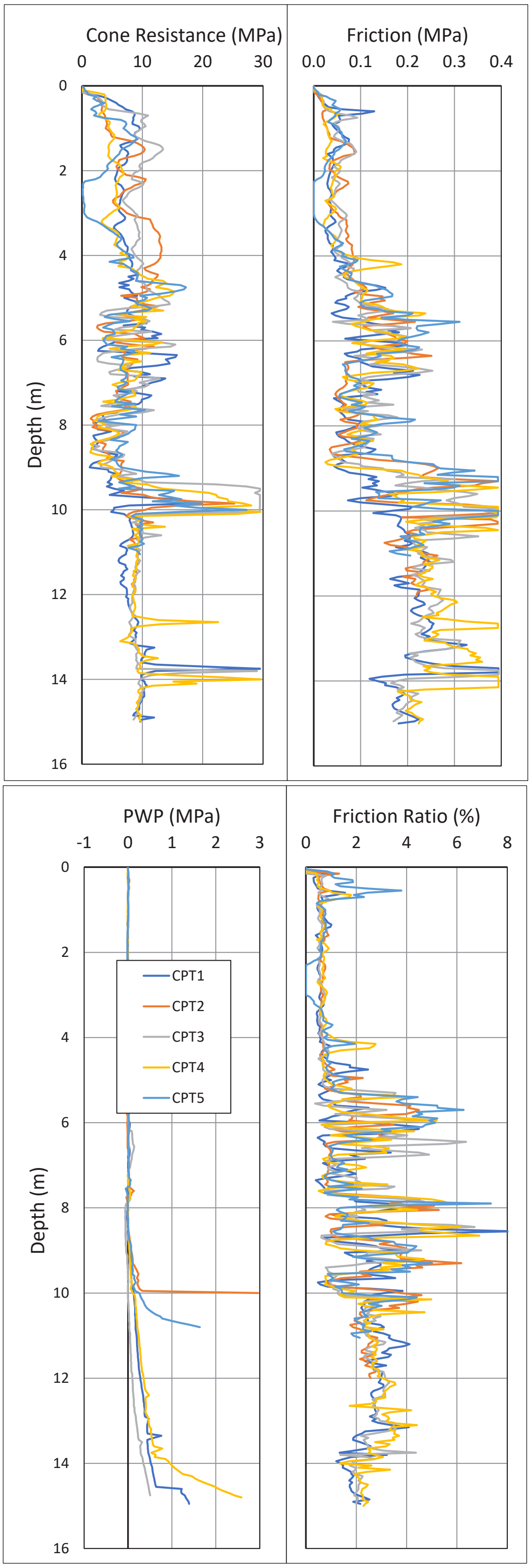

Figure B6. SAND: CPTu tests.