M. B. Jaksa *et al.* (2005). *Géotechnique* **55**, No. 2, 109–121

Towards reliable and effective site investigations

M. B. JAKSA*, J. S. GOLDSWORTHY*, G. A. FENTON†, W. S. KAGGWA*, D. V. GRIFFITHS‡, Y. L. KUO* and H. G. POULOS§

It is widely appreciated that, in civil engineering and building projects, the largest element of financial and technical risk usually lies in the ground. Almost exclusively, the scope of geotechnical investigations is governed not by what is needed to characterise the subsurface conditions appropriately but, rather, by how much the client and project manager are willing to spend. There is often little correlation between the variability of the ground and the scope of the investigation. This paper presents the results of a Monte Carlo simulation incorporating many 3D single-layer soil profiles with different statistical characteristics. A three-storey building founded on nine pad footings is used to assess the reliability of various site investigation scopes and test methods. The pad footings are designed on the basis of settlement, and are examined using 3D finite element analysis and Schmertmann's method. It is observed, as expected, that the likelihood of underdesigning or overdesigning a footing decreases as the scope of the investigation increases. The relationship between these likelihoods and the variability of the ground is presented.

KEYWORDS: footings/foundations; numerical modelling and analysis; site investigation; statistical analysis;

Il est largement accepté que, pour les projets de génie civil et de construction, l'élément de risque financier et technique le plus important touche généralement au sol. Presque exclusivement, la portée des enquêtes géotechniques est gouvernée non pas par ce qui est nécessaire pour caractériser des conditions de sous-surfaces appropriées mais plutôt par les sommes que le client et le chef de projet sont disposés à dépenser. Il existe souvent peu de corrélation entre la variabilité du sol et la portée de l'investigation. Cet exposé présente les résultats d'une simulation de Monte Carlo incorporant de nombreux profils de sol unicouche en 3 dimensions avec diverses caractéristiques statistiques. Nous utilisons un bâtiment de trois étage fondé sur neuf assises coussin pour évaluer la fiabilité de diverses portées d'investigation du site et diverses méthodes d'essai. Les assises sont conçues sur la base d'affaissement et sont examinées en utilisant une analyse d'élément fini en 3 dimensions et la méthode de Schmertmann. Nous observons, comme on pouvait s'y attendre, que la probabilité de sous-concevoir ou de surconcevoir une assise diminue à mesure que la portée de l'investigation augmente. Nous présentons la relation entre ces probabilités et la variabilité du sol.

INTRODUCTION

The scope of geotechnical site investigations is rarely related to the anticipated variability of the ground, and is often chosen to minimise initial costs (Institution of Civil Engineers, 1991). Many studies over the last 15 years or so have clearly demonstrated that, in civil engineering and building projects, the largest element of financial and technical risk often lies in the ground and, as a result, minimum initial cost investigations can lead to significant cost over-runs and delays during construction (National Research Council, 1984; Institution of Civil Engineers, 1991; Littlejohn *et al.*, 1994; Whyte, 1995). Expenditure on geotechnical site investigations varies considerably, being sometimes as low as between 0·025% (Jaksa, 2000) and 0·3% (National Research Council, 1984) of the total project cost. Site investigations that inadequately quantify the variability of the ground can result in three possible cost outcomes:

(*a*) The foundation is underdesigned as a result of an overly optimistic geotechnical model, and hence fails to comply with the design criteria, which can ultimately lead to some level of structural distress.

(*b*) The foundation is overdesigned as a consequence of a pessimistic geotechnical model and/or inherent conservatism in the design process.

(*c*) Unforeseen conditions require substantial changes to the foundation system, which also result in construction delays.

This has led the Institution of Civil Engineers (1991) to conclude that: 'You pay for a site investigation whether you have one or not.'

There currently exists little guidance in relation to determining the scope of an appropriate investigation for a given site, other than in the form of generic, non-site-specific rules of thumb (Lowe & Zaccheo, 1991; Bowles, 1996). This paper investigates, using 3D numerical simulations, the effect of soil variability and site investigation scope on the design and subsequent performance of a three-storey building founded on nine pad footings and located on a site with plan dimensions of 50 m × 50 m. Two site investigation strategies are examined: one based on discrete sampling, as in the case of the standard penetration test (SPT), and the other based on continuous sampling, as would occur with a cone penetration test (CPT). Several numbers of boreholes and soundings are investigated in order to determine the optimal site investigation strategy for sites with different soil variability characteristics. It is demonstrated that, as expected, additional sampling yields better estimates of footing size up to a certain number of boreholes, beyond which, additional sampling and testing provide marginal improvement in footing size.

Manuscript received 14 May 2004; revised manuscript accepted 19 November 2004.

Discussion on this paper closes on 1 September 2005, for further details see p. ii.

* School of Civil and Environmental Engineering, University of Adelaide, Australia.

† Department of Engineering Mathematics, Dalhousie University, Halifax, Canada.

‡ Division of Engineering, Colorado School of Mines, Golden, USA.

§ Coffey Geosciences Pty Ltd, New South Wales, Australia.

APPROACH

Basic philosophy

It is never possible to know the geotechnical properties at every location beneath an actual site because, in order to do so, one would need to sample and/or test the entire subsurface profile. However, it is possible to generate 3D subsurface profiles, by means of spatial variability simulation, whose spatial characteristics mimic those of actual soils. (The process of spatial variability simulation will be discussed later.)

Consider a realisation of a 3D soil mass, obtained from a spatial variability simulation model. The subsurface profile is first discretised into a series of elements, e.g. 0·5 m × 0·5 m × 0·5 m in size, as in a regular 3D finite element mesh, and different geotechnical properties are assigned to each element, depending on the nature of the spatial variability of the soil profile. Since the soil mass has been simulated, its properties are known completely at every location, or element. Using 3D finite element analysis (FEA), and given a loading condition, it is possible to determine the most appropriate, or optimal, footing to satisfy a set of design criteria, since the properties of the soil mass are exactly known.

Juxtapose the above idealisation with the usual situation encountered in practice, where the soil properties are known only at a limited number of discrete locations, as a result of in-situ tests or field sampling and subsequent laboratory testing. If one then designs a footing based on this incomplete knowledge, using the same loading and design criteria, it is then possible to compare this 'traditional' design with the 'optimal' design obtained above from complete knowledge of the site. If the traditionally designed footing is larger than the 'optimal' footing, then it is overdesigned, as a consequence of the incomplete knowledge derived from an inadequate site investigation programme. If, on the other hand, the traditional footing is smaller than the optimal, then it fails to comply with the design criteria. In this case, the structure will require some level of refurbishment work, ranging from minor repairs to total demolition and rebuild, depending on how much smaller the traditional footing is in relation to the footing that is actually required (i.e. the optimal footing).

If a site is generated many hundreds of times, within a Monte Carlo framework, it is possible to derive probabilities of underdesign and overdesign for a variety of site investigation programmes. It is then possible to compare the efficiency of one sampling programme with another, and, if one incorporates costs into this framework, one is able to determine the true financial benefit or penalty of adopting a more exhaustive site investigation scheme. For example, is it better to specify four boreholes with standard penetration tests (SPTs) at 1·5 m intervals, four cone penetration tests (CPTs), or six CPTs?

Details of analysis

The framework adopted in this paper is summarised in flowchart form in Fig. 1, and is discussed below. Jaksa *et al.* (2003) describe the framework in greater detail.

In order to implement this procedure, first a site is generated using random field theory (e.g. Vanmarcke, 1983), which makes use of three statistical properties: the mean, a measure of the variance (e.g. standard deviation, coefficient of variation (COV)), and the *scale of fluctuation*, θ, which expresses the correlation of properties with distance. A large value of θ, for a particular soil property, implies a soil mass where the property fluctuates with distance slowly about the mean, suggesting a more continuous soil mass, whereas a small θ implies that the property fluctuates rapidly about the mean, suggesting a more randomly varying soil mass. Random field theory is implemented in the analyses performed below using the local average subdivision technique developed by Fenton & Vanmarcke (1990) and Fenton (1994). As mentioned above, a single-layer 3D soil profile beneath a building site is simulated. In the case of the analyses presented in this paper, the site is nominally 50 m × 50 m in plan by 50 m deep. Each element represents a block of soil 0·5 m × 0·5 m × 0·5 m in size, and hence the soil profile consists of 100 × 100 × 100 elements, yielding a total of one million elements. The spatial statistics of the simulated soil profile conform to the specified mean, COV and θ. The values of θ adopted are consistent with those published in the literature (e.g. Vanmarcke, 1977; Fenton & Vanmarcke, 1991; Wickremesinghe & Campanella, 1993; Jaksa, 1995; Jaksa *et al.*, 1993, 1997, 1999, 2004; Cafaro *et al.*, 1999). It is common for θ to be anisotropic: that is, θ in the two orthogonal horizontal directions, θ_H, is larger than that in the vertical direction, θ_V. This is due to the geological processes that form soils, which typically result in greater variation in the vertical direction than horizontally. As a starting point, this paper examines soil profiles with $\theta_H/\theta_V = 1$ and 2. Single-layer soil profiles with small (random)

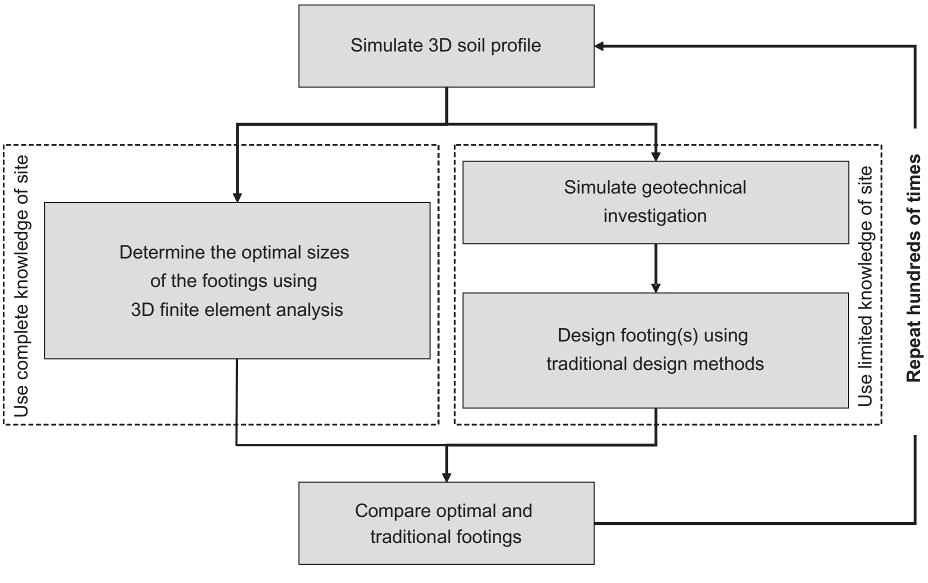


Fig. 1. Flowchart of adopted approach

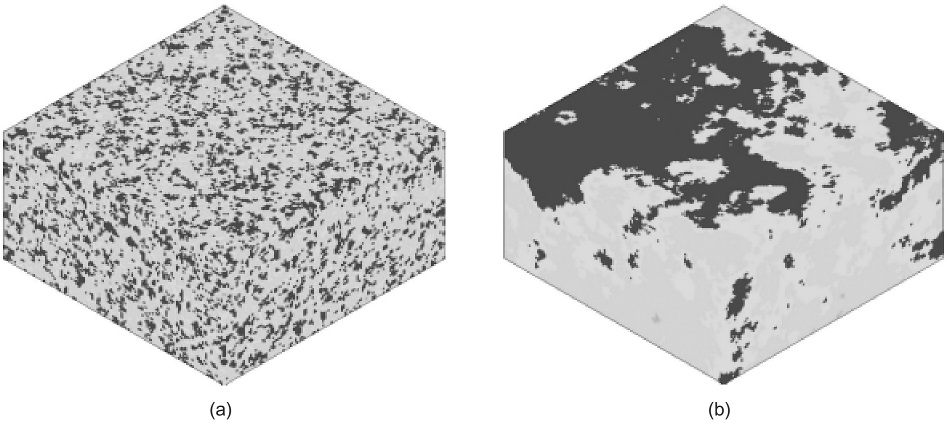
and large (continuous) scales of fluctuation, θ, with identical means and COVs, are shown isometrically in Fig. 2(a) and 2(b) respectively. In these realisations, lighter colours represent stiffer soil.

Although foundation design is an iterative process, balancing uncertainties, load factors, ground strengths and so forth, the essential features of the process can be captured, in the first instance, by restricting attention to elastic-like settlements in otherwise sufficiently strong ground. This then makes the spatial variation in elastic modulus, E, the only parameter of interest in what follows. The analyses presented below hence involve the determination of the elastic settlement of a regular nine-pad footing, with each footing being subjected to a single vertical concentrated load located at its centre. The value of Poisson's ratio, ν, is assumed to be constant, and equal to 0·3 irrespective of location, as it has been found that it has a much smaller influence on settlement behaviour than E and is, typically, not that variable. A total of 25 different single-layer soil profiles are examined, each with a mean elastic modulus of 30 MPa. The profiles investigated include combinations of COVs equal to 10%, 20%, 30%, 40% and 50%, and scales of fluctuation (θ_H, θ_V), in metres, equal to (1, 1), (2, 1), (4, 2), (8, 4) and (16, 8). It is noted that the same value of θ_H is adopted for both orthogonal horizontal directions. Also, to strictly enforce that non-negative values of E are generated, a lognormal distribution of E is adopted (Fenton & Griffiths, 2005).

The structural details are shown in Fig. 3. The building footprint is a 20 m square, and the centre-to-centre spacing of the footings is 8 m in both directions. To simulate loads consistent with a three-storey building, a dead load of 5 kPa and a live load of 3 kPa is applied to each floor, which corresponds to a central concentrated vertical force of 1536 kN (8 m × 8 m tributary area × 8 kPa × 3 storeys) being applied to the centre footing, 864 kN (6 m × 6 m tributary area × 8 kPa × 3 storeys) being applied to each of the four corner footings, and 1152 kN (8 m × 6 m tributary area × 8 kPa × 3 storeys) to each of the four middle-edge footings. A maximum allowable total settlement of 25 mm for each footing is specified, and a maximum allowable differential settlement of 0·0025 m/m between footings (Day, 1998). In addition, the base of each pad is located on the ground surface, and the thicknesses of the pad footings are not examined.

Once the subsurface profile has been simulated, each of the one million elements can be used in designing the nine pad footings, using 3D FEA to satisfy the specified loading and settlement constraints. Each of the nine footings is designed separately and can adopt different plan areas. Hence the resulting system of nine pad footings is optimal for the simulated site and design constraints. The finite element model used in the analyses is based on the random finite element model code *rsetl3d* developed by Fenton & Griffiths (2005). In this model the footings are assumed to be rigid, and footing rotation is not permitted.

Owing to the discretisation of the finite element mesh, the process of determining the optimal footings proceeds in the manner shown in Fig. 4. First, in order to minimise numerical errors associated with the finite element method, for each of the nine pads a minimum footing size of 3 × 3 elements (1·5 m × 1·5 m in plan) is examined (Fig. 4(a)). The loads are applied and the total and differential settlements calculated. If the total and/or differential settlements of any of the nine footings are greater than those allowed, the plan areas of the violating footings are increased. In order to maintain a central point load on all footings, each



(a) (b)

Fig. 2. 3D soil profiles: (a) small θ (random soil profile); (b) large θ (continuous soil profile)

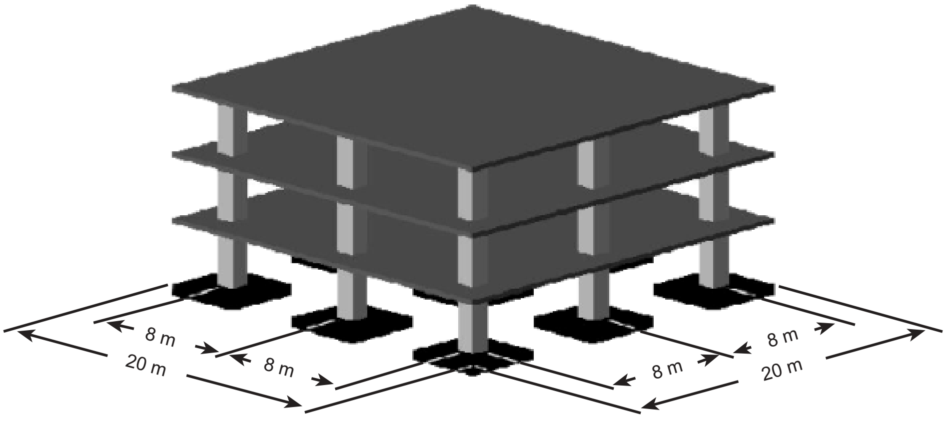


Fig. 3. Layout of three-storey, nine-pad footing building examined

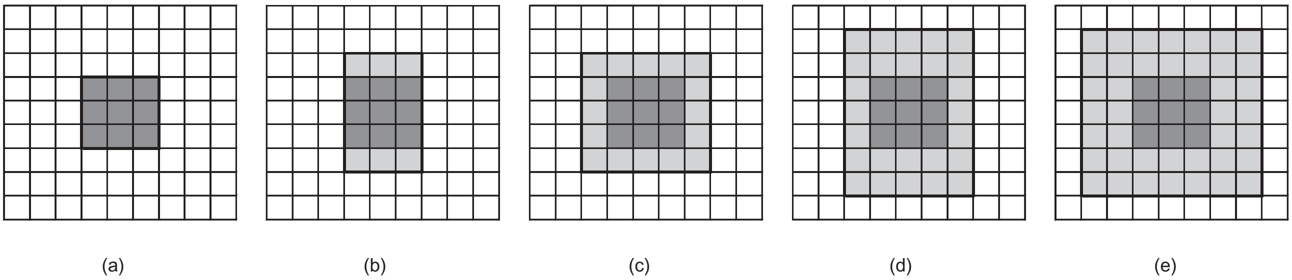


Fig. 4. Process of determining footing dimensions (plan view of ground surface)

area is increased by six elements, three either side of the original nine-element footing (Fig. 4(b)). If the design criteria again fail to be satisfied, the area of each violating footing is further enlarged, as shown in Fig. 4(c)–(e). The footing areas continue to be enlarged about the original nine-element footing until the design criteria are satisfied. An inherent limitation of such an approach is that the optimal footing is determined in discrete, and sometimes relatively large, increments. As a result, the size of the optimal footing is not known with great precision. However, this is not unlike the actual situation where footings are increased in size by discrete steps of 0·25 or 0·5 m, for example. There is, of course, a trade-off between the precision of the optimal footing size and the number of elements included in the finite element mesh. Although a finer mesh will result in the size of the optimal footing being known with greater precision, a significant increase in computational time will also occur.

In order to model the process of conventional footing design practice, a site investigation is simulated. In this paper, two testing schemes are examined. First, samples are taken or tests are performed at discrete vertical intervals of 1·5 m, as would occur with the SPT or triaxial tests. This is achieved by sampling every third element along a vertical ‘boring’ or transect. Second, a continuous sounding is simulated, as would occur with the CPT, where every element along a vertical ‘boring’, or transect, is intercepted. In addition, 12 site investigation schemes are investigated, as shown in Fig. 5. The solid line surrounding each strategy represents the building footprint (20 m × 20 m), which is located centrally on the 50 m × 50 m site, and the solid circles represent the boreholes or soundings. As shown in Fig. 5, the 12 site investigation schemes that are examined incorporate between 1 and 25 boreholes in a regular grid pattern: the adopted nomenclature refers to the investigation number, regular grid (‘RG’) pattern and the total number of boreholes. The purpose of the 12 schemes is to investigate the value of performing an increased scale of sampling and testing, as well as the location of the boreholes themselves. Future studies will examine the optimal location and pattern of boreholes in greater detail.

In order to compare the SPT and CPT investigations appropriately, it is necessary to include testing errors. It is widely recognised that the SPT is associated with much greater testing uncertainty than the CPT. This is due to equipment, procedural and operator-induced uncertainties (Orchant *et al.*, 1988; Jefferies and Davies, 1993). Lee *et al.* (1983) suggested that the COV for the SPT varies between 27% and 85%, whereas Phoon & Kulhawy (1999) suggested that it varies between 25% and 50%. On the other hand, Orchant *et al.* (1988) recommended that the COV for the CPT varies between 7% and 12%, whereas Phoon & Kulhawy (1999) found that it varies between 5% and 40% in clays. Studies by Jaksa (1995) suggest a much lower value. Hence, in the analyses described below, the following values are adopted: $\mathrm{COV_{SPT}} = 50\%$ and $\mathrm{COV_{CPT}} = 20\%$. Note that these values account solely for measurement error, and do not include spatial variability or model uncertainty, the latter accounting for errors associated with the relationship between the measured parameter and the derived soil property, e.g. SPT N and E. The measurement errors are applied as an additional uncertainty derived from a uniform distribution. For example, once each test location is determined, and the simulated values of E are obtained from these locations, the measurement uncertainty ($\mathrm{COV_{SPT}}$ or $\mathrm{COV_{CPT}}$) is added to account for testing error.

As most footings that are designed in practice do not involve 3D FEA, the settlements are calculated using the commonly adopted approach developed by Schmertmann (1978). Hence the footings are designed based on the limited knowledge obtained from the simulated geotechnical investigations and implemented by Schmertmann’s method. In order to design the nine footings more appropriately, the influence of adjacent footings is included in the settlement calculation of each by evaluating the resulting additional stresses using the Boussinesq equations and adopting the relationships described by Holtz (1991). In addition, in order to integrate Schmertmann’s strain influence factor triangle

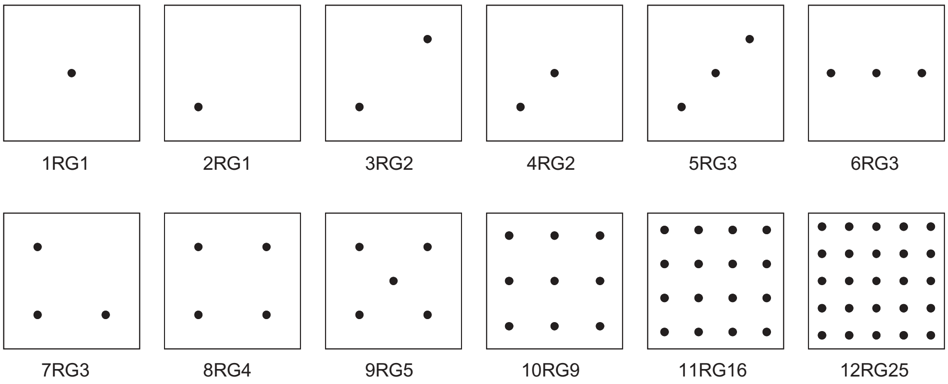


Fig. 5. Site investigation schemes examined

more precisely, a vertical step size of 0·01 m is used, rather than the 0·5 m element size.

The optimal and traditionally (Schmertmann) designed footings are then compared as outlined previously. Typically, the process described above, for a single soil profile and 1000 realisations, takes approximately 150 h to converge to a solution on a supercomputer using eight simultaneous processors, with each processor handling single realisations in turn.

RESULTS AND DISCUSSION

In order to illustrate the variability associated with different scales of fluctuation, Fig. 6 shows the results of the

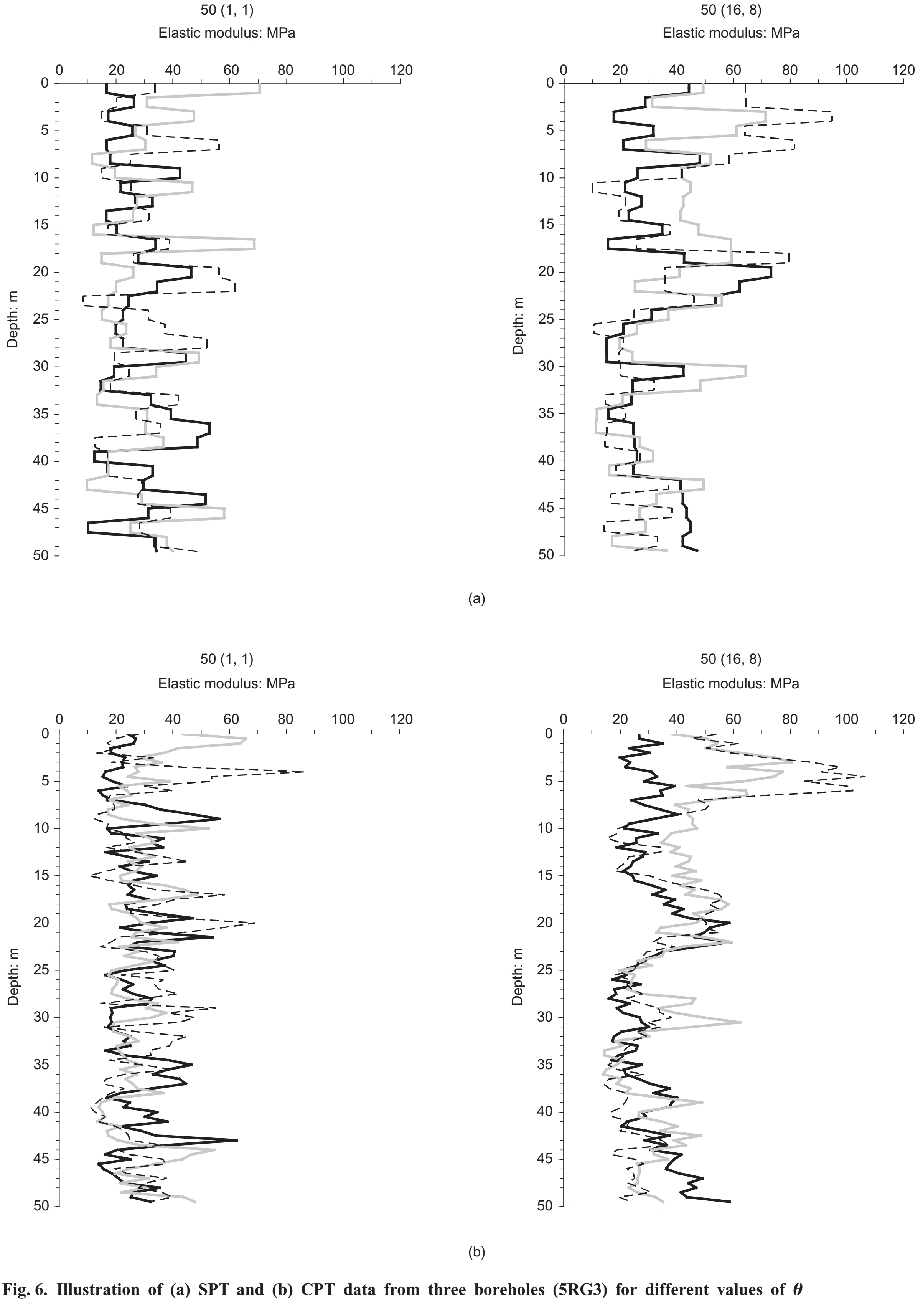


Fig. 6. Illustration of (a) SPT and (b) CPT data from three boreholes (5RG3) for different values of θ

simulated site investigation associated with scheme 5RG3. Each plot shows the results, for a soil variability COV of 50%, of the three boreholes obtained from a single random field realisation. The left-hand plots refer to a realisation with $(\theta_H, \theta_V) = (1, 1)$ and the right-hand plots to $(\theta_H, \theta_V) = (16, 8)$. The upper two plots show the results of SPTs at 1·5 m vertical intervals and the lower two the results of 'continuous' CPTs, with data spaced at 0·5 m vertical intervals. Note that these results include testing errors of $COV_{SPT} = 50\%$ and $COV_{CPT} = 20\%$, as discussed above, which are superimposed on the variability of the soil profile itself, i.e. COV = 50%. It is evident from this figure, as explained previously, that small values of θ result in more erratic or rapid fluctuations.

Finite element analyses used for complete site knowledge

Figure 7 summarises the results of four analyses in terms of the mean total footing area (i.e. the sum of the areas of the nine footings averaged over the 1000 realisations) as a function of the number of boreholes. These traditional analyses are confined solely to the SPT. The legend identifying each of the four curves refers, first, to the soil variability COV, as a percentage, and second to the horizontal and vertical scales of fluctuation, respectively, in metres. Note that, for reasons that will be explained later, the results of the optimal footing design derived from 3D FEA are not shown on this plot. Instead, for each of the curves there is an associated dashed line, which refers to the result of the optimal footing design that is obtained if complete knowledge (CK) of the site is used in the application of Schmertmann's method. In this case, for each of the nine footings, the optimal footing is designed based on each element of soil within the region associated with each footing. The region adopted is 8 m × 8 m in plan (i.e. the centre-to-centre spacing of the footings) × 2*B* m in depth (*B* being the footing width), as is consistent with Schmertmann's method. (Although this region seems appropriate, future studies will investigate the significance of varying its extent.)

Figure 7 shows that, as expected, as the extent of the site investigation increases, the mean footing design area decreases, for both of the COVs presented. Perhaps surprisingly, the more continuous soil profiles [10 (16, 8) and 50 (16, 8)] yield greater variation, with increasing site investigation extent, than the more randomly fluctuating profiles [10 (1, 1) and 50 (1, 1)]. This is particularly apparent with the 50 (16, 8) case. (This will be discussed in greater detail later.) Furthermore, it can be seen in each case that, as a greater number of boreholes is adopted, the footings derived from the site investigation converge asymptotically towards those obtained from complete knowledge of the site (the dashed lines), as one would expect.

A similar behaviour is observed when one examines the standard deviation of the total footing area as a function of the extent of the site investigation, as shown in Fig. 8. Again, for larger numbers of boreholes, the standard deviation of the footings derived from the site investigation approaches that associated with the footings designed from complete knowledge of the site—again, as expected.

Figure 9 presents the results of the probabilities of over- and underdesign as a function of the number of boreholes. Overdesign is defined as the situation, for any realisation, where the total area of the nine footings derived from the site investigation data and Schmertmann's method exceeds the total footing area designed on the basis of complete knowledge of the site and 3D FEA. Underdesign is the reverse situation, where the total area of the nine footings derived from the site investigation data and Schmertmann's method falls below the total footing area designed on the basis of complete knowledge of the site and 3D FEA. Again, Fig. 9 suggests that the amount of overdesign decreases as the extent of the investigation increases. However, Fig. 9

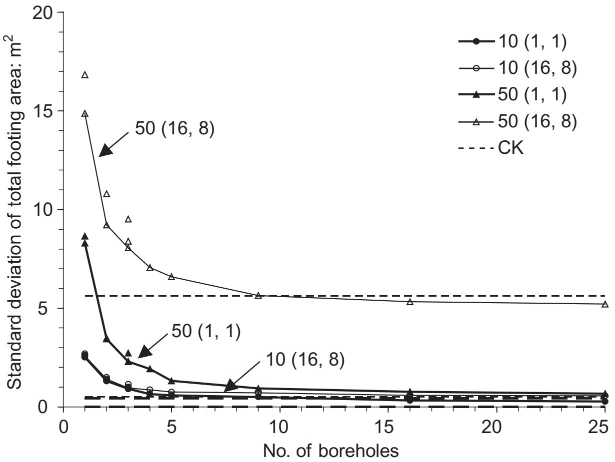


Fig. 8. Standard deviation of footing size against extent of site investigation based on Schmertmann's method and SPTs

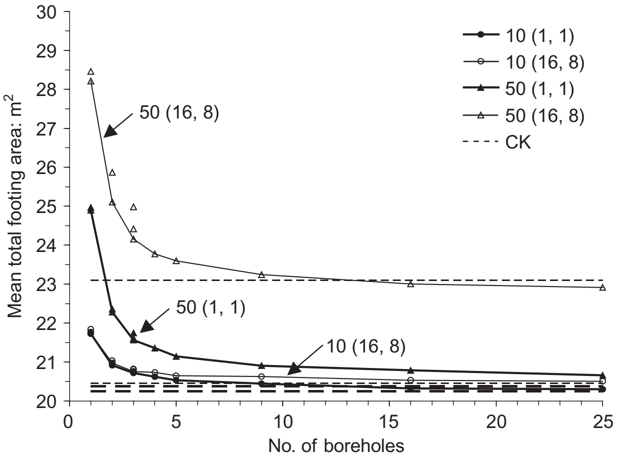


Fig. 7. Mean footing size against extent of site investigation based on Schmertmann's method and SPTs

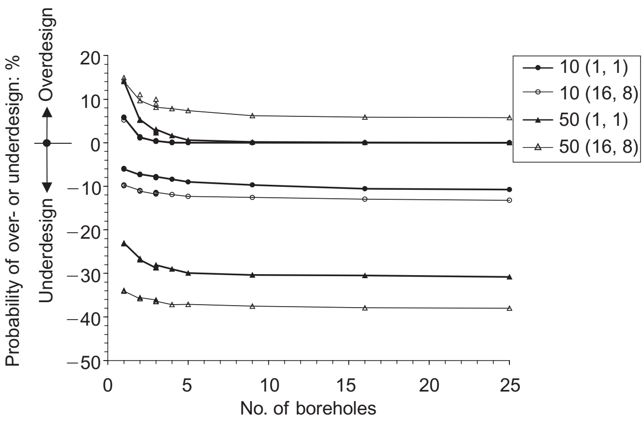


Fig. 9. Probability of over- and underdesign for footings designed using SPT data and Schmertmann's method (when compared with footings designed using complete knowledge and FEA)

also shows that the probability of underdesign increases as the extent of the investigation increases. This is contrary to what one would expect, but can be explained by the process adopted. As the optimal footing is based on a 3D FEA of the complete site, and the traditional footing design is based on the Schmertmann method, Fig. 9 includes model uncertainty, as explained earlier, which is the error between the finite element and Schmertmann techniques. For example, the mean optimal footing for the 50 (16, 8) case is 39·44 m^2, whereas if an optimal footing design was performed with knowledge of the entire site, using Schmertmann's method rather than 3D FEA, the mean optimal footing size would have been 23·10 m^2. As a result, even with complete knowledge of the site, the traditional design will always differ from the optimal footing obtained using 3D FEA, in this case by 16·34 m^2. To complicate matters further, the results presented in Fig. 9 are likely to be influenced, to some degree, by the adoption of the minimum footing size of 2·25 m^2 for the finite element analyses, as described previously.

For each of the cases examined, this discrepancy with respect to the mean and standard deviation of the total footing areas (i.e. the sum of the nine footing areas examined over the ensemble of 1000 realisations) is given in Table 1. This table demonstrates that, as the COV and θ increase, so too do the mean and standard deviations of the total footing area. In order to explain such behaviour, one needs to examine the nature of variability in more detail. Imagine two soil profiles, one with a small scale of fluctuation, e.g. (1, 1), which exhibits rapid variability over small distances, and the other with a larger scale of fluctuation, e.g. (16, 8), which is more continuous and varies slowly with distance, as shown previously in Figs 2 and 6. It is common for the latter, continuous profile to include larger pockets of soft or stiff material (Fig. 2(b)). As footing settlement essentially involves averaging the properties of many elements of soil within its zone of influence, the variability of settlement-based footing designs increases as the scale of fluctuation increases. This is because footings founded on continuous profiles might be founded on large zones of soft or stiff material, as well as material of intermediate stiffness, hence resulting in a wider range of possible footing sizes when compared with rapidly fluctuating profiles where the zone of influence effectively averages out the variability.

Schmertmann's method used for complete site knowledge

In order to compare more appropriately the footing sizes obtained, the remainder of the paper will make use of a slightly modified procedure from that described above. Rather than designing the optimal footings using 3D FEA, they will be designed using Schmertmann's method, adopting the process described earlier. As 3D FEA is not used in these subsequent analyses, the incremental increases in footing area are no longer constrained by the finite element mesh of 0·5 m × 0·5 m, as shown previously in Fig. 4. Instead, each individual footing will be increased in width by 0·1 m (0·05 m either side of the footing's centre). This enables more precise footing areas to be established. In addition, the minimum footing size of 1·5 m × 1·5 m that was used previously in the FEAs to maintain reliable numerical estimates is no longer required.

Figures 10 and 11 present the results for both SPT- and CPT-based site investigations, respectively, for COVs of 10% and 50% and scales of fluctuation of (1, 1) and (16, 8) obtained from 1000 realisations. (In comparison with the 3D FEAs, 1000 such realisations take approximately 2 h to converge to a solution on a single-processor desktop computer.) As can be seen from these two figures, the proportions of overdesign and underdesign generally decrease with a greater number of boreholes and as the COV decreases. In

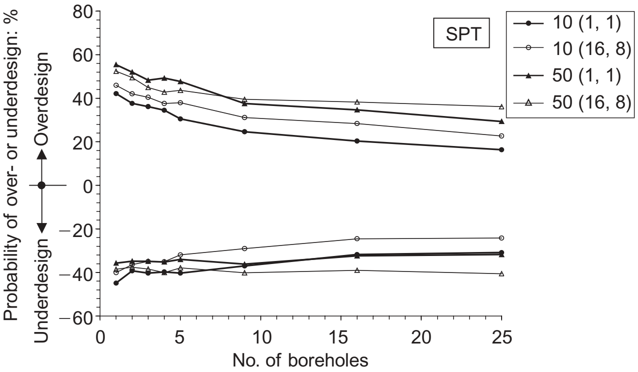


Fig. 10. Probability of over- and underdesign for footings designed using SPT data and Schmertmann's method (when compared with footings designed using complete knowledge and Schmertmann's method)

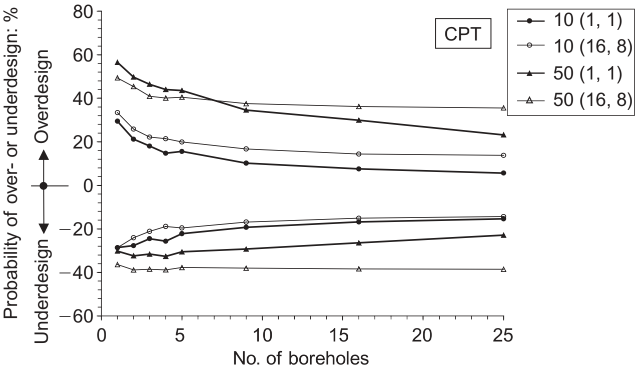


Fig. 11. Probability of over- and underdesign for footings designed using CPT data and Schmertmann's method (when compared with footings designed using complete knowledge and Schmertmann's method)

Table 1. Means and standard deviations of total footing areas designed using Schmertmann's and finite element methods with complete knowledge of the site for different soil variabilities

Soil variability COV (θ_H, θ_V)	Mean: m^2		Standard deviation: m^2	
	Schmertmann	Finite element	Schmertmann	Finite element
10 (1, 1)	20·25	21·76	0·00	0·18
10 (16, 8)	20·45	22·87	0·51	1·92
50 (1, 1)	20·39	26·70	0·43	1·84
50 (16, 8)	23·10	39·44	5·62	25·52

addition, the SPT (Fig. 10) yields a greater proportion of overdesign and underdesign than the CPT (Fig. 11), as expected. However, the more continuous soil profiles, with scales of fluctuation of 16 m in the horizontal direction and 8 m in the vertical, in general yield a greater proportion of overdesign and underdesign than the more randomly fluctuating profiles, i.e. (1, 1). The reason for this, as explained above, is related to the 'averaging' effect associated with footing settlement, which results in greater footing size variability as the scale of fluctuation increases.

It is also evident from Figs 10 and 11, for the more continuous profiles, that there is only marginal benefit in increasing the scope of investigation beyond approximately five boreholes. This is particularly so for the 50 (16, 8) case. This is because, owing to their continuity, a small number of boreholes will adequately quantify their variability. Drilling

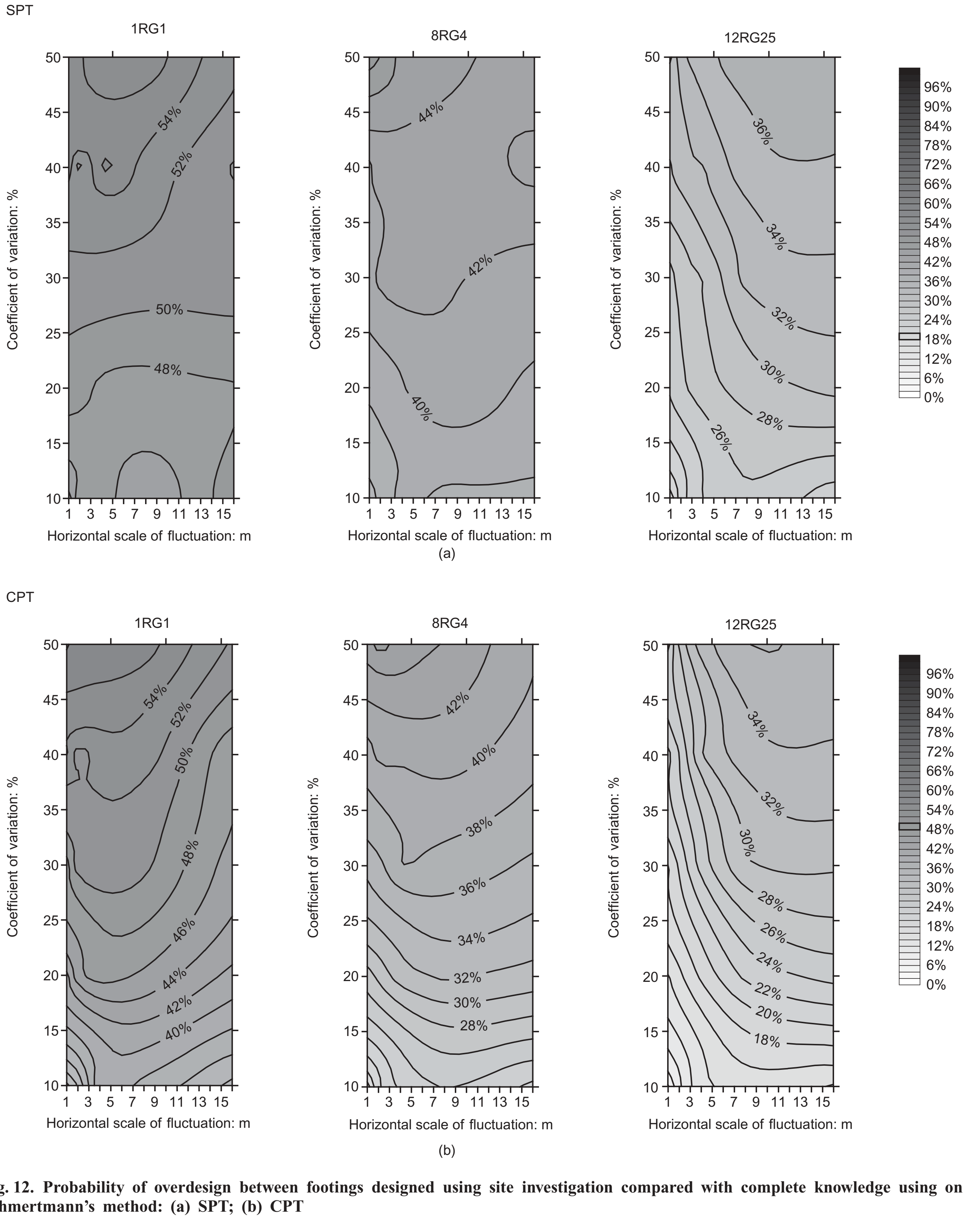


Fig. 12. Probability of overdesign between footings designed using site investigation compared with complete knowledge using only Schmertmann's method: (a) SPT; (b) CPT

further boreholes will thus result in redundant information being obtained.

In contrast to Figs 7–9, Figs 10 and 11 show the results of only one scheme for each number of boreholes. Recall from Fig. 5 that the following multiple schemes were examined: one borehole, 1RG1, 2RG1; two boreholes, 3RG2, 4RG2; three boreholes, 5RG3, 6RG3 and 7RG3. Figs 10 and 11 show the results of only one scheme for each borehole: that is, the one associated with the smallest value of over- or underdesign. This is because, from the limited number of runs performed, no consistent pattern emerged in relation to the best of these schemes. As mentioned pre-

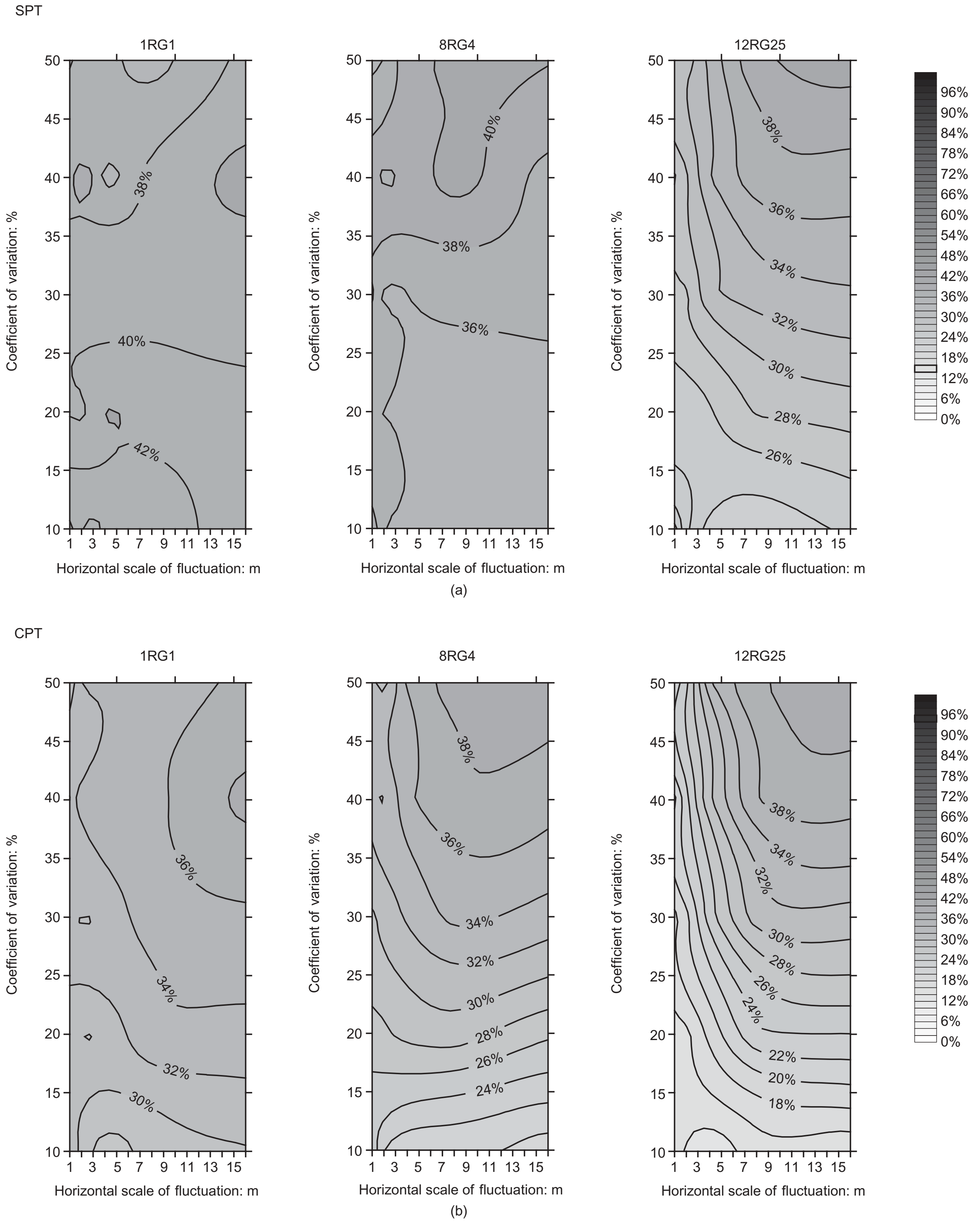
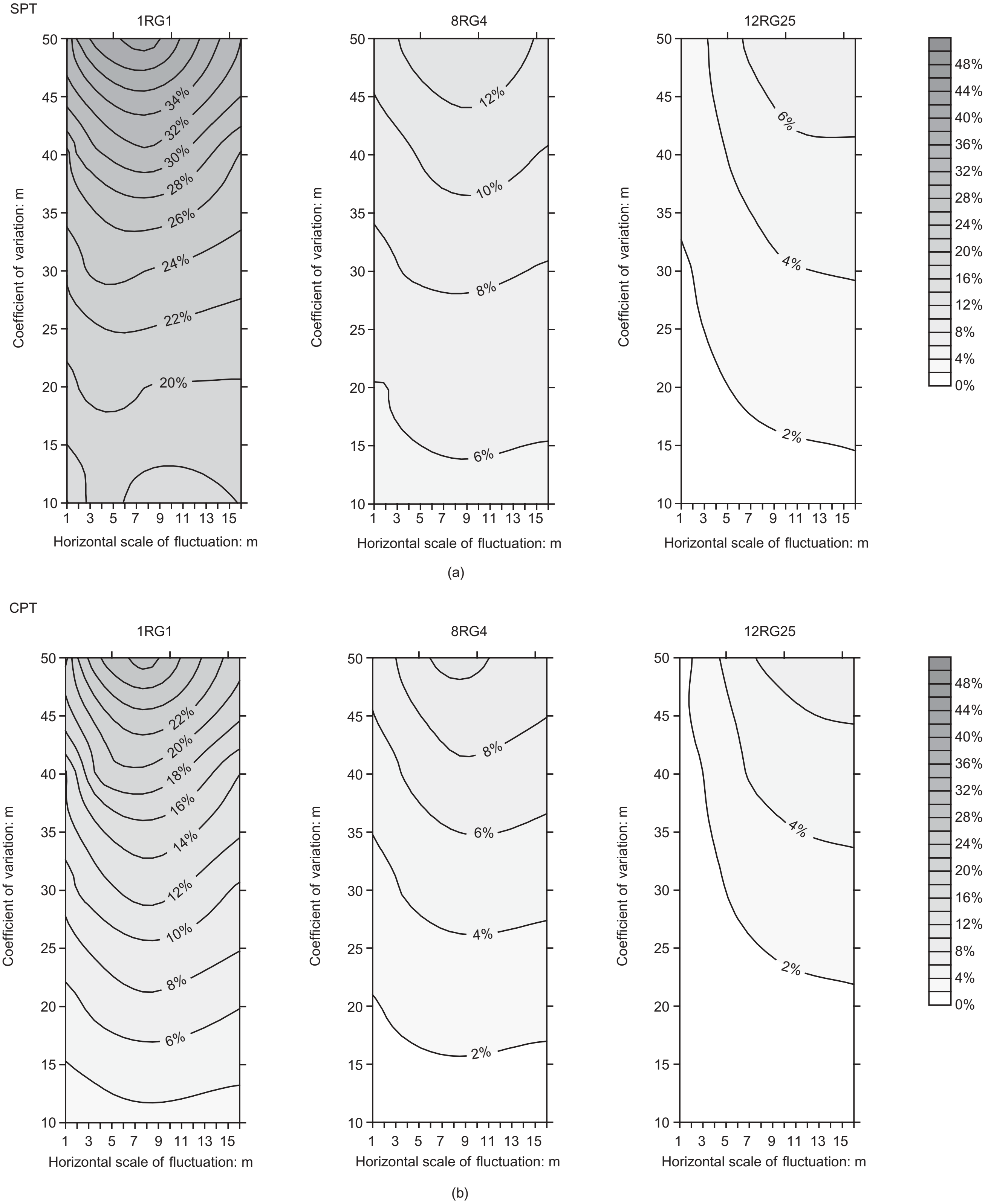


Fig. 13. Probability of underdesign between footings designed using site investigation compared complete knowledge using only Schmertmann's method: (a) SPT; (b) CPT

viously, future studies will examine these patterns in more detail.

Owing to the number of combinations of COV and θ examined, it is possible to generate contour plots of the relationship between COV, θ and the probability of overdesign and underdesign for investigations based on the SPT and CPT. Figures 12 and 13 present the probabilities of overdesign and underdesign, respectively, obtained by comparing footings designed using data obtained from a site-specific investigation with those derived from complete knowledge of the site. It is clear from these figures that the proportion of underdesign and overdesign reduces considerably as the extent of the investigation increases. This is also the case as the COV decreases, and less so as the scale of fluctuation de-



(a)

(b)

Fig. 14. Mean error difference between footings designed using site investigation compared with complete knowledge when an overdesign occurs using only Schmertmann's method: (a) SPT; (b) CPT

creases. In addition, site investigations based on the CPT yield a lower proportion of underdesigned or overdesigned footings when compared with those derived from SPT-based site investigations.

Figures 14 and 15 show, as a function of COV and θ, the mean percentage error between footings designed using site investigation data and those designed using complete knowledge of the site when either an underdesign or overdesign occurs, respectively. This is calculated, for example in the underdesign case, by summing the relative area difference between the underdesigned and optimal footings, and dividing by the total number of underdesigned footings. As one would expect, these plots show relationships similar to those exhibited in Figs 12 and 13. However, they more readily

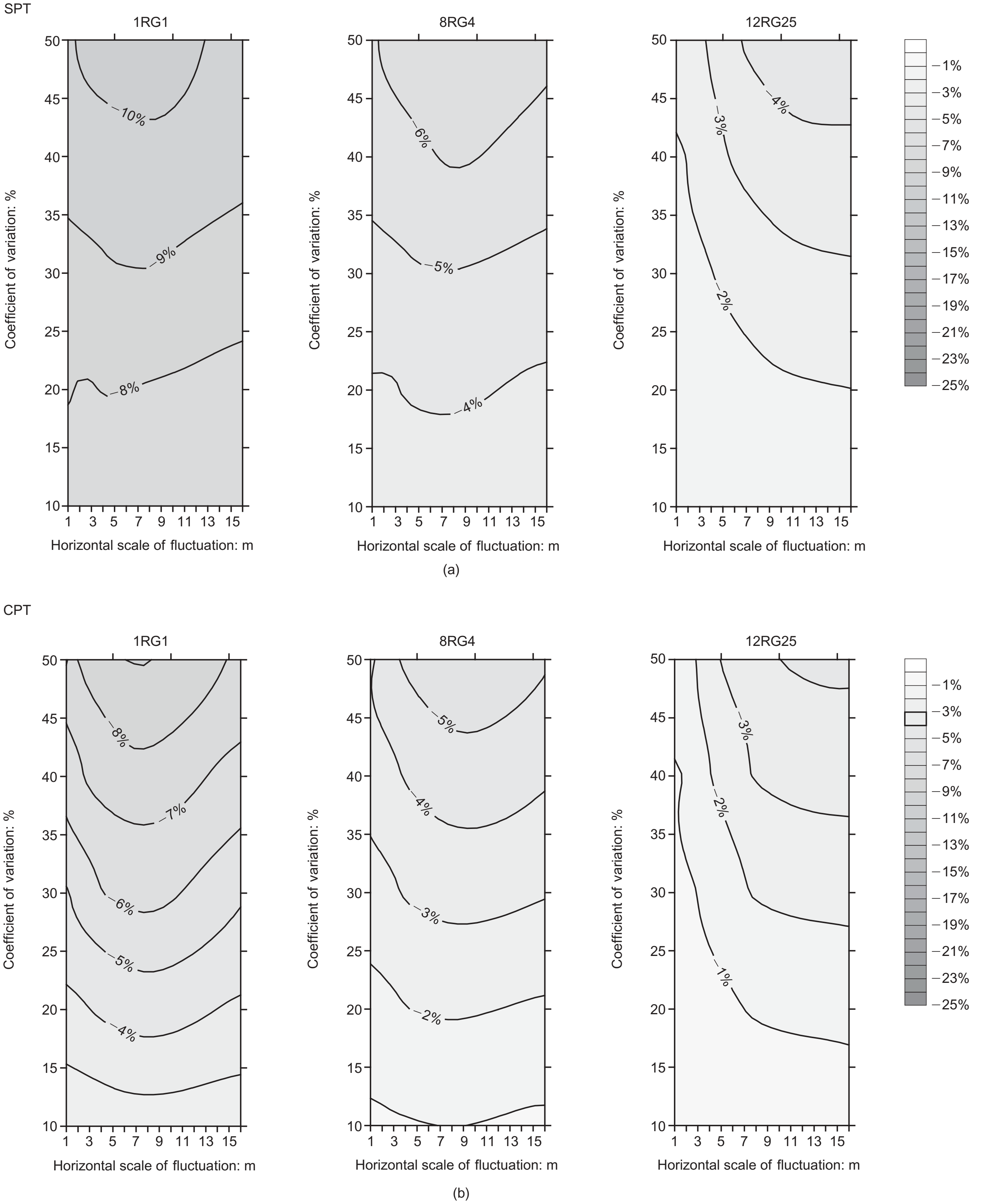


Fig. 15. Mean error difference between footings designed using site investigation compared with complete knowledge when an underdesign occurs using only Schmertmann's method: (a) SPT; (b) CPT

highlight, particularly for 1RG1 and 8RG4, that a worst case appears to be associated with a horizontal scale of fluctuation of 8 m. This distance corresponds to the spacing of the columns: that is, the centre-to-centre spacing of the footings. Fenton & Griffiths (2004) investigated the differential settlement associated with two footings, and also found that the worst case θ was associated with the centre-to-centre spacing of the footings. This is a convenient observation, because, if the scale of fluctuation for a particular site is unknown, rather than needing to measure it—which can be very costly and time-consuming—one can assume a worst-case value equivalent to the centre-to-centre spacing of the footings.

Further analyses are needed before generic and comprehensive site investigation guidelines can be developed. Future studies should incorporate costs, other footing types, such as rafts, and other test types, including triaxial tests and the flat dilatometer test. In addition, sampling patterns and multi-layer profiles should also be investigated. Preliminary work on the relationship between the cost of foundation failures and the scope of site investigations has been published by the authors (Goldsworthy *et al.*, 2004). The long-term objective is to develop a series of guidelines that will enable geotechnical engineers to compare and discuss, with the client, the ramifications and cost-effectiveness of several geotechnical investigation scenarios. In this way the client will be better informed in relation to the risk of foundation failure and overdesign associated with the adopted site investigation, which, it is hoped, will lead to more reliable and cost-effective site investigations.

CONCLUSIONS

Preliminary results have been presented describing the relationship between the extent of site investigations and the variability of a single-layer soil profile, based on a nine-pad footing system where only settlements have been considered. A framework proposed by Jaksa *et al.* (2003) has been implemented, and it has been observed that, not unexpectedly, the probability of underdesigning or overdesigning a footing decreases as the scope of the investigation increases. However, there will come a point where additional site investigation expenditure will not improve the reliability of the footing system, which will be highlighted when costs are included. A 'worst case' scale of fluctuation has been observed that coincides with the centre-to-centre spacing of the footings. Hence, if the scale of fluctuation for a particular site is unknown, one can assume a conservative value equal to the centre-to-centre spacing of the footings.

ACKNOWLEDGEMENTS

The authors wish to thank the reviewers for their contributions and suggestions, which have significantly added to the clarity and value of the paper. The research presented in this paper was funded by the Australian Research Council (No. DP0211195), and their support is gratefully acknowledged. In addition, *rsetl3d* was developed with the assistance of a National Sciences and Engineering Research Council of Canada Discovery Grant (No. RGPIN0105445) and a National Science Foundation of the United States of America Grant (No. CMS-9877189), and the authors are grateful for their support. Any opinions, findings, conclusions or recommendations are those of the authors and do not necessarily reflect the views of the aforementioned organisations.

REFERENCES

Bowles, J. E. (1996). *Foundation analysis and design*, 5th edn. New York: McGraw-Hill.

Cafaro, F., Cherubini, C. & Cotecchia, F. (1999). Use of the scale of fluctuation to describe the geotechnical variability of an Italian clay. *Proc. 8th Int. Conf. Application of Statistics and Probability, Sydney* **1**, 481–486.

Day, R. W. (1998). *Forensic geotechnical and foundation engineering*. New York: McGraw Hill.

Fenton, G. A. (1994). Error evaluation of three random field generators. *ASCE J. Engng Mech.* **120**, No. 12, 2478–2497.

Fenton, G. A. & Griffiths, D. V. (2005). Three-dimensional probabilistic foundation settlement. *J. Geotech. Geoenviron. Engng, ASCE* **131**, No. 2, 232–239.

Fenton, G. A. & Vanmarcke, E. H. (1990). Simulation of random fields via local average subdivision. *ASCE J. Engng Mech.* **116**, No. 8, 1733–1749.

Fenton, G. A. & Vanmarcke, E. H. (1991). Spatial variation in liquefaction risk assessment. *Proceedings of the geotechnical engineering congress*, ASCE, Boulder, CO, pp. 594–607.

Goldsworthy, J. S., Jaksa, M. B., Kaggwa, W. S., Fenton, G. A., Griffiths, D. V. & Poulos, H. G. (2004). Cost of foundation failures due to limited site investigations. *Proceedings of the international conference on structural and foundation failures, Singapore*, pp. 398–409.

Holtz, R. D. (1991). Stress distribution and settlement of shallow foundations In *Foundation engineering handbook*, 2nd edn (ed. H. Y. Fang, pp. 166–222. New York: Van Nostrand Reinhold.

Institution of Civil Engineers (1991). *Inadequate site investigation*. London: Thomas Telford.

Jaksa, M. B. (1995). *The influence of spatial variability on the geotechnical design properties of a stiff, overconsolidated clay.* PhD thesis, Faculty of Engineering, University of Adelaide.

Jaksa, M. B. (2000). Geotechnical risk and inadequate site investigations: a case study. *Aust. Geomech.* **35**, No. 2, 39–46.

Jaksa, M. B., Kaggwa, W. S. & Brooker, P. I. (1993). Geostatistical modelling of the undrained shear strength of a stiff, overconsolidated, clay. *Proceedings of the conference on probabilistic methods in geotechnical engineering*, Canberra, pp. 185–194.

Jaksa, M. B., Brooker, P. I. & Kaggwa, W. S. (1997). Inaccuracies associated with estimating random measurement errors. *J. Geotech. Geoenviron. Engng, ASCE* **123**, No. 5, 393–401.

Jaksa, M. B., Kaggwa, W. S. & Brooker, P. I. (1999). Experimental evaluation of the scale of fluctuation of a stiff clay. *Proc. 8th Int. Conf. Appl. Stat. Probabil., Sydney* **1**, 415–422.

Jaksa, M. B., Kaggwa, W. S., Fenton, G. A. & Poulos, H. G. (2003). A framework for quantifying the reliability of geotechnical investigations. *Proc. 9th Conf. Appl. Stat. Probabil. Civ. Engng, San Francisco* **2**, 1285–1291.

Jaksa, M. B., Yeong, K. S., Wong, K. T. & Lee, S. L. (2004). Horizontal spatial variability of elastic modulus in sand from the dilatometer. *Proc. 9th Aust. NZ Conf. Geomech., Auckland* **1**, 289–294.

Jefferies, M. G. & Davies, M. P. (1993). Use of the CPTu to estimate equivalent SPT N_{60}. *ASTM Geotech. Test. J.*, **16**, No. 4, 458–468.

Lee, I. K., White, W. & Ingles, O. G. (1983). *Geotechnical engineering*. Boston: Pitman.

Littlejohn, G. S., Cole, K. W. & Mellors, T. W. (1994). Without site investigation ground is a hazard. *Proc. Inst. Civ. Engrs Civ. Engng* **102**, No. 2, 72–78.

Lowe, J. III & Zaccheo, P. F. (1991). Subsurface exploration and sampling. In *Foundation engineering*, 2nd edn (ed. H.-Y. Fang), 1–71. New York: Chapman & Hall.

National Research Council (1984). *Geotechnical site investigations for underground projects*, US National Committee on Tunnelling Technology, Vol. 1. Washington: National Academy Press.

Orchant, C. J., Kulhawy, F. H. & Trautmann, C. H. (1988). *Critical evaluation of in-situ test methods and their variability*, Report EL-5507, Vol. 2. Palo Alto: Electric Power Research Institute.

Phoon, K. K. & Kulhawy, F. H. (1999). Characterization of geotechnical variability. *Can. Geotech. J.* **36**, No. 4, 612–624.

Schmertmann, J. H. (1978). *Guidelines for cone penetration test: performance and design*, FHWA-TS-78-209, US Dept of Transportation. Washington, DC: Federal Highway Administration.

Vanmarcke, E. H. (1977). Reliability of earth slopes. *J. Geotech. Engng Div.*, ASCE **103**, No. 11, 1247–1265.

Vanmarcke, E. H. (1983). *Random fields: Analysis and synthesis*. Cambridge: MIT Press.

Whyte, I. L. (1995). The financial benefit from site investigation strategy. *Ground Engng*, **28**, October, 33–36.

Wickremesinghe, D. & Campanella, R. G. (1993). Scale of fluctuation as a descriptor of soil variability. *Proceedings of the conference on probabilistic methods in geotechnical engineering*, Canberra, pp. 233–239.