# General report for TC 205
# Safety and serviceability in geotechnical design: a reliability-based perspective

## Rapport général du TC 205
## Sécurité et maintenance en conception géotechnique : une perspective fiabiliste

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ABSTRACT: A transition has been underway in geotechnical engineering from design based on analyses that treat problems as if every pertinent variable were deterministic only to recognize at the end that uncertainties exist by using a factor of safety to design based on probability concepts. In probabilistic design, key problem variables are treated, at least implicitly, as random variables, and probabilities of undesired outcomes (defined through the concept of limit states) are kept below certain maximum acceptable values. The ways to perform these analyses are many, depending on choices as to, for example, which variables to treat as random and which method of analysis (e.g., the limit equilibrium method, the finite element method) to use to solve the underlying boundary-value problem. Different approaches have been followed in North American and Europe, the two regions where use of probabilistic, limit states-based design is more common. Many unresolved issues remain. This paper highlights some of these issues in the context of papers submitted on the topic to the 18th ICSMGE.

RÉSUMÉ: Une transition s'est déroulée en géotechnique depuis une conception basée sur des analyses qui traitent les problèmes comme si chaque variable pertinente était déterministe vers la prise en compte des incertitudes par l'utilisation de facteurs de sécurité dans le cadre d'une conception basée sur des concepts probabilistes. Dans ce cadre probabiliste, les variables clés du problème sont traitées, au moins implicitement, comme des variables aléatoires et les probabilités des résultats attendus (définis par le concept d'état limite) sont maintenues en-dessous certaines valeurs maximales acceptables. Il y a de nombreux moyens de réaliser ces analyses, selon le choix, par exemple, des variables à traiter comme aléatoires et des méthodes d'analyse utilisée pour régler le problème aux limites considéré (e.g., la méthode des équilibres limites ou les éléments finis). Différentes approches ont été suivies en Amérique du Nord et en Europe, les deux régions où l'utilisation des méthodes probabilistes aux états-limites sont les plus répandues. De nombreuses questions restent posées. Cet article met en évidence ces questions dans le contexte des articles soumis au 18$^{ème}$ CIMSG sur ce thème.

KEYWORDS: stability, serviceability, geotechnical design, Eurocode 7, load factors, resistance factors, partial factors, characteristic and design values.

## 1 BACKGROUND

Development of the Eurocode started a large-scale effort to address geotechnical engineering problems in a more systematic manner from the point of view of the uncertainties present in these problems. One major theme of this effort has been to allocate uncertainties more precisely, e.g., between loads and resistances or, in the case of the Eurocode, between loads and the quantities that enable calculation of soil resistances. Another important theme has been to transition away from *ad hoc* measures of safety to probabilities of failure, with failure defined rather specifically. This emphasis on probability of failure has started an important debate concerning what probabilities of failure are acceptable given project importance and consequences of failure.

In Europe, the path followed was to account for uncertainties by dividing shear strengths or, more generally, soil properties controlling the specific resistance being calculated by material factors typically greater than one. Actions (defined in the code as loads or applied displacements) would be increased by factors also greater than one. This general approach is usually referred to as a partial factors approach. In North America, a different philosophy developed. In the still-evolving North American practice, an approach called Load and Resistance Factor Design (LRFD) is used. The main difference with respect to European practice is that design is based on the following inequality:

$$(RF)R_n \geq \sum (LF_i)Q_{i,n} \qquad (1)$$

where $R_n$ is the nominal resistance, RF is the corresponding resistance factor, $Q_{i,n}$ are nominal loads (with dead load, live load, etc., identified each by a different value of i), and $LF_i$ are the corresponding load factors.

Both the Eurocode and its North American counterparts are based on the concept of Limit-States Design (LSD). Limit states exist on the boundary between acceptable and unacceptable states. They offer a solid conceptual basis for the definition of failure, which then becomes the achievement of an undesirable state as defined by the limit states. The probability of failure then becomes nothing more than the probability of achievement of an undesirable state. Figure 1 shows a hypothetical design problem involving two variables, resistance and load, both of which are random variables. The line in the figure separates acceptable pairings of Q and R (Q < R) from those that would lead to failure. The line is the locus of limit states, and states above the line are failure states.

The advantage of use of inequality (1) over the European approach is that the resistance factor reflects all of the uncertainties involved in calculating resistance, including uncertainties in both soil property determination and the analysis used to calculate the resistance. This advantage is likely more of a conceptual nature, for it is possible to account for analysis uncertainty also in the European approach, but that must be done through the material factors dividing the strength parameters, which cease to be then pure factors on material properties.

Another important difference between Eurocode-related efforts and development of North American LRFD is in the process. The Eurocode started with an exclusive focus on buildings (Simpson and Driscoll 1998) but then developed into a code with broad application in geotechnical design. In North America, particularly in the United States, the effort targeted transportation infrastructure, leaving state DOTs and individual researchers with the task of developing load and resistance factors that made sense and could be used in a practical manner. This has in a way been an incentive for performing research in the subject.

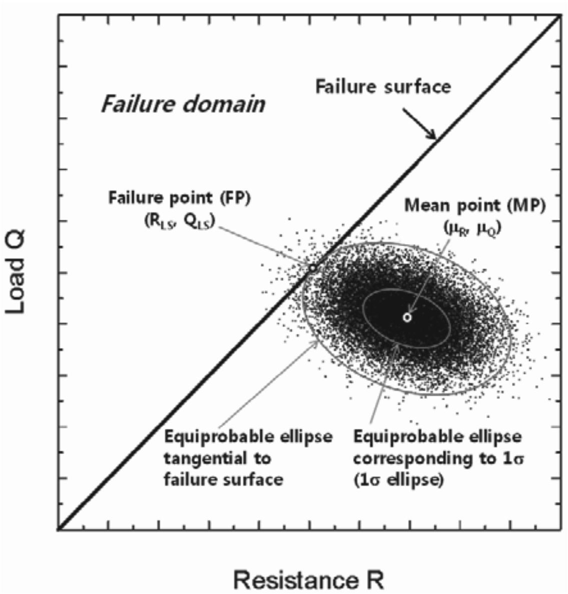


Figure 1– Design problem with load and resistance being ramdom variables (after Salgado and Kim 2013).

The topic of safety and serviceability in geotechnical engineering, which involves design methods and approaches for a variety of geotechnical problems based on analyses that account for probabilities and variability in pertinent variables, is complex and should evolve over the years with the development of better methods of analyses and a greater understanding of variability in geotechnical problems. The following sections summarize current knowledge and challenges, together with the contributions of papers submitted to the XVIII ICSMGE.

## 2 RELIABILITY ANALYSIS

Research on limit-states design (LSD) in its various forms has ranged from simple calibrations of material factors (in the Eurocode framework) or resistance factors (in the LRFD framework) so that results would match those obtained using traditional methods with accepted values of factors of safety to reliability analysis performed with various levels of sophistication. The tools to perform sophisticated reliability analysis have evolved considerable in the last 20 years, and most advances that will be useful to code designers in the future are likely to result from this type of analysis.

Figure 1 shows a cloud of points representing pairings of load Q and resistance R around the mean point ($\mu_R$,$\mu_Q$) of these two variables. Each of these points is a simultaneous realization of random variables R and Q. Since Q and R are more likely to be close to their respective mean, there is a heavier concentration of points near ($\mu_R$,$\mu_Q$); however, if the load and resistance in a hypothetical problem deviate from their means by sufficiently large amounts, Q may end up exceeding R, which would, by definition, constitute failure. This cloud of points may be obtained using Monte Carlo simulations if the probability distributions of Q and R are known. The probability of failure can be estimated from the ratio of the number *n* of points above the limit state line shown in the figure to the total number *N* of points. Mathematically:

$$p_f = \lim_{N\to\infty} \frac{n}{N} \qquad (2)$$

where *N* must be large enough for $p_f$ to converge.

Also shown in Figure 1 are dispersion ellipses. A dispersion ellipse represents the locus of the resistance-load pairings with the same level of deviation from the mean. Note that there is one dispersion ellipse that is tangent to the failure line at a point known as the most likely "failure point" FP, also known as the "design point". From the point of view of development of material factors or resistance factors, if the probability of failure calculated for the simulations of Figure 1 is the value that should be targeted in design, the relationship between the design point FP and the mean point MP yields directly the factors that would be used in design. This is easiest to show for LRFD, in which case the factors RF and LF shown in inequality (1) would follow from:

$$\mathrm{RF} = \frac{\mathrm{R_{LS}}}{\mu_\mathrm{R}} \quad \text{and} \quad \mathrm{LF} = \frac{\mathrm{Q_{LS}}}{\mu_\mathrm{Q}} \qquad (3)$$

where $R_{LS}$ and $Q_{LS}$ are the resistance and load at the design point FP; and $\mu_R$ and $\mu_Q$ are the means of resistance and load.

For different reasons, code designers may set load factors $LF_i^*$ that must apply to a range of design settings. If a resistance factor is developed using reliability analysis (calculated from an equation like (3)), this resistance factor is consistent with the load factor calculated using the same equation. If it is to be used with a different load factor specified in a code, it must be adjusted. If we refer to the adjusted resistance factor as $RF^*$, it may be computed by requiring that inequality (1) apply equally whether $RF^*$ and $LF_i^*$ or RF and $LF_i$ are used:

$$\mathrm{RF}^* = \mathrm{RF}\frac{\sum(\mathrm{LF}_i^*)\mathrm{Q}_{i,n}}{\sum(\mathrm{LF}_i)\mathrm{Q}_{i,n}} \qquad (4)$$

## 3 CODE DESIGN

The main goal of modern LSD-based codes is to prevent the occurrence of limit states or, more precisely, to guide or prescribe designs so as to keep the probability of such occurrence acceptably small. The concepts presented in Figure 1 apply equally to ultimate (ULS) and serviceability (SLS) limit states.

One of the key points in connection with LRFD and with partial factor design if model (analysis) uncertainty is included in the factors is that factors are specific to a given analysis. It is not possible to use resistance factors in an *ad hoc* manner or the purpose of LRFD or partial factor design is lost. This is a significant culture change for geotechnical engineers, for factors of safety have traditionally been used with considerable flexibility. This significant difference between practice as it is evolving at present and the practice of years back has important implications. It becomes difficult, for example, to use a method of analysis or design for which factors have not been developed. There are also ethical and professional conduct implications, as touched upon by Redaelli (2013), who points out that ethics should constrain engineers from selecting factors of safety based on financial motivations. In geotechnical engineering, Redaelli (2013) argues, the high level of uncertainty invites a reliance on judgment, which in turn enables geotechnical engineers willing to deliberately distort design outcomes for their own interests or for the interest of their direct employer to easily do so. Redaelli (2013) believes that honest and scrupulous engineers in a consulting company may meet with resistance from clients and colleagues when attempting to incorporate into their design uncertainty that others may wish to ignore. It can be argued that codes containing partial factors or load and resistance factors determined carefully and scientifically would be of assistance in minimizing this problem.

Limit states design codes have not been and could not be based primarily on reliability analysis and research, largely because the research was not available. Many challenges remain, chiefly among them a proper assessment of soil variability both for code design and for specific projects. Although it is a stated goal of modern limit states codes to use acceptable probability of failure as a design reference and a basis to develop values of materials or resistance factors, the present reality is that codes have been largely developed based on calibrations, as noted by Länsivaara and Poutanen (2013), and code development committee deliberations. As results of better, more specific research become available, code designers may be able to take advantage of these results to modify values and recommendations in future editions of design codes.

Although this paper focuses on safety and serviceability as addressed by codes and design calculations, not every risk can be quantified; some are best avoided. For example, blunders can be avoided by having layers of checks on procedures and calculations put in place. Other risks may be avoided by taking a broad view of the project, looking for things that can go wrong and adapting designs and construction procedures based on this assessment. Robert (2013) discusses this type of project-management approach to risk management.

The next sections examine specific aspects of the design of slopes, foundations and retaining structures using modern limit states codes.

## 4 STABILITY OF SLOPES

### 4.1 *General Remarks*

Stability limit states, also known as ultimate limit states, are associated with dangerous outcomes. In mechanics terms, they would often be associated with collapse. A loss of slope stability would be associated with large slope movements after driving actions overcame available resistance. This is the primary design check for slopes, although more contained slope movement that would not be analyzed in the same manner are also sometimes critical.

Loads are, for the most widely used slope stability analysis methods, expressed through driving moments. The driving moment due to dead loads (self-weight of a potential sliding mass or permanent external loads acting on the boundary of the sliding mass) may be denoted by $M_{d,DL}$. The driving moment due to live loads (nonpermanent loads on the crest of the slope, such as vehicular loads) may be denoted by $M_{d,LL}$. Resistances are expressed through a resisting moment $M_r$. In terms of driving and resisting moments, inequality (1) becomes:

$$(RF)M_{rn} \geq (LF_{DL})M_{d,DL}\big|_n + (LF_{LL})M_{d,LL}\big|_n \qquad (5)$$

Studies on probabilistic stability analysis of slopes and embankments started in the early 1970s and have continued (e.g., Cornell 1971, Tang et al. 1976, Christian et al. 1992). In early slope reliability analysis studies, the first-order second-moment method was popular for the assessment of probability of slope failure. More recent studies have made use of the first-order reliability method (FORM) and Monte Carlo simulations (MCSs) to assess the probability of slope failure (Christian et al. 1994, Tobutt 1982).

In probabilistic slope stability analysis, it is essential to consider the spatial variability of the soil in the slope. This is the most significant shortcoming of original studies on this topic. If a soil slope is assumed to remain homogenous when soil properties are varied (say, using Monte Carlo simulations) in an analytical study, the reality that these properties in fact vary with some degree of independence across the slope is completely ignored, and probabilities of failure calculated in this manner will not be accurate. Salgado and Kim (2013) have used an advanced FORM method, coupled with random field modeling of the slope, to develop resistance factors for use in slope stability as a function of soil spatial variability. Although the results have been consolidated in terms of resistance factors, it would be possible to develop partial material factors from their reliability analyses. The fact that model uncertainty in slope stability analysis is small to negligible (Kim et al. 1999, Kim et al. 1992, Yu et al. 1998) makes it relatively easy to obtain Eurocode-like material factors from reliability analyses such as those performed by Salgado and Kim (2013).

As pointed out by Länsivaara and Poutanen (2013), use of (5) or another inequality like it requires the slope stability software to provide the values of moments from permanent, temporary and other loads separately. Although not yet common, this will not be an impediment for long (STABL WV, for example, is a slope stability software that already provides this level of information detail).

A key decision in performing reliability-based design or developing resistance or material factors is what the acceptable probability of failure $p_f$ is. Christian (2004) characterized risk as a function of number of fatalities, referring to various efforts by regulators and others. Most slopes would be well designed with $p_f = 10^{-3}$, but lower values of $p_f$ might be needed for structures whose failure would lead to large economic or human loss.

### 4.2 *Papers*

Länsivaara and Poutanen (2013) review the prescriptions of the Eurocode (EN-1997) regarding slope stability. The Eurocode prescribes three different ways of checking stability: design approaches DA1, DA2 or DA3.

According to these authors, DA1 with combination 2 and DA3 are most commonly used for slope stability analysis. There are also three different reliability classes (RC1, RC2 and RC3), which allow the accounting for consequences of attainment of a limit state, into which a slope would be fit before analyses are done. Another way allowed by the code to account for consequences of attainment of a limit state is to increase material factors for cases with more consequential failures.

As seen earlier, material strength is divided by a material factor, and this is supposed to account for uncertainties in shear strength and any analysis uncertainty. The material factor for effective stress analyses for soils modeled as Mohr-Coulomb materials is 1.25; for a total stress analysis of a slope with soil modeled as a Tresca material, it is 1.4. Self-weight is left unfactored but live (temporary) loads (actions) on the slope are factored by 1.3. Länsivaara and Potanen (2013) suggest that there is an "overestimation of safety" implied in typical factors of safety used in total stress analysis of the stability of clay slopes. Salgado and Kim (2013) also observed this possibility in the results of reliability analysis of slopes modeled using random fields.

Lechwicsz and Wrzesiński (2013) report on field-scale staged construction of an embankment on top of an organic soil deposit, the last stage built up until failure occurred. The shear strength of the soil profile on which the embankment was built was characterized using results of the vane shear test corrected according to the Swedish Geotechnical Institute method (Larsson et al. 1984) with correction factors determined based on triaxial compression (TC), triaxial extension (TE) and direct simple shear (DSS) testing of the materials in the laboratory. The shear strength data determined in this manner allowed estimation of the probability distributions of the undrained shear strength. Their analysis of the results of these field experiments focused on testing the Eurocode 7 DA1, DA2 and DA3 design approaches.

The embankment was constructed in three stages on organic subsoil from 1983 to 1987. The structure was then brought to failure by increasing the height of the fill (Wolski et al. 1988, 1989). The organic soil at the site, present as two separate layers (a 3.1-m-thick amorphous peat layer and a 4.7-m-thick calcareous-organic soil layer) is underlain by a sand layer. The water table is located near the ground surface, but the sand layer was subjected to artesian conditions, with the piezometric level as much as 1.5 meters above the ground surface. Effective stresses *in situ* were in the single digits (in units of kPa). The OCR of the organic soils was estimated to decrease from 5 near the surface to 2 at depth. The undrained shear strength was determined both before and after construction of the embankment. After consolidation, the strength was observed to increase below the embankment, with larger increases near the center. Interestingly, there seems to have been a slight reduction, on average, of the coefficients of variation of the shear strength from the values before embankment construction (all except one greater than 0.11 and as high as 0.19) after the embankment was constructed and consolidation completed; originally, these values were as low as 0.03, with several observations below 0.1. Consolidation may have evened out some spatial variability initially present.

The stability analysis performed for the third stage and the failure test assumed the organic subsoil to be divided into three different shear zones: A – below the embankment crest, B – below the embankment slope and C – to the side of the embankment. In these analyses, which relied on the Bishop Simplified Method, various approaches to account for uncertainty in shear strength were considered: direct use of mean values, use of characteristic values determined as the mean less 0.5 or one times the standard deviation, and application or not of a partial factor $\gamma_m = 1.25$ to the mean and characteristic values of shear strength. This value is not the 1.4 prescribed by the Eurocode for Tresca materials, but the authors did not elaborate.

Based on comparisons between these different analyses and the results of the test embankment, the authors recommended use of design approach DA1 with combination 1 and use of characteristic values of undrained shear strength defined as a conservative mean (mean less a fraction of the standard deviation) for slope stability analysis under similar conditions. Design Approaches DA1 with combination 2 and DA3 were deemed excessively conservative.

One of the most challenging aspects of probabilistic stability analyses of slopes is that the spatial nature of the variability of slopes must be taken into account in order to produce reasonable results, as noted earlier. Lechwicsz and Wrzesiński (2013) attempt to assess, using judgment, the degree of conservativeness of different approaches to determination of characteristic strengths and partial factors to use in design, but that appears to be a difficult task to complete until more rigorous analyses that fully consider spatial variability of the slope are properly done and scrutinized. The test to failure of the embankment slope that they describe is precisely the type of field result needed to combine with the rigorous reliability analyses done today to validate theoretical work. More information on spatial variability, which is difficult to assess, particularly in the horizontal direction, is certainly needed in field tests of this type.

## 5 FOUNDATIONS

### 5.1 *General Remarks*

Foundation engineering problems are interaction problems: a foundation element (e.g., a footing or a pile) or a collection of foundation elements interact with the soil around them through the interface between the elements and the soil. The presence of this interface can be exploited in probabilistic analyses of foundation limit states: the variability, spatial in nature, of soil properties result in contact stresses at this interface that are a function of the response of the entire soil mass. It is customary to observe (directly measure or deduce from instrumentation using load cells and strain gauges) the values of these boundary loads on foundation elements, and the variability of these stresses can be used, combined with specific methods of analysis, to perform reliability analyses. This approach, as already discussed, is not viable for slope stability analysis.

Basu and Salgado (2012), for example, performed such an analysis for drilled shafts in sand. They placed the analyses of Loukidis and Salgado (2008) and Lee and Salgado (1999) on a probabilistic basis and used Monte Carlo simulations to determine the resistance factors corresponding to probability of failure equal to $10^{-3}$ and $10^{-4}$. Rigorous reliability analysis combined with realistic and appropriate soil models, with a rigorous method of analysis of a particular boundary-value problem, and with careful accounting of variability of every random variable entering the calculation produces excellent and transparent results. The body of work of this type is still limited, but it is likely to produce results that will be useful to code designers.

Differently from slopes, which are most often checked for stability, deflection-based limit states, whether potentially leading to ultimate or serviceability limit states, are more frequent for foundations than a classical foundation plunge limit state. In more complex structures, such as piled rafts, analyses are more involved, often requiring numerical solutions, which makes a reliability analysis more challenging and makes it more difficult to consider spatial variability indirectly.

One of the hardest decisions for engineers to make in the context of reliability-based design is on the acceptable probability of failure. In every geotechnical design problem, the answer can be different. For example, in connection with pile foundations used in traditional solutions (not in a piled raft), a possible way to think of probability of failure $p_f$ would be to pose a question such as "how often would an engineer be willing to deal with cracking of the superstructure (an indicator of a possible ULS) to keep initial foundations cost low?" A possible answer would be that no more often than would be the case if one pile in ten thousand settled too much, which could then lead to an acceptable probability of failure of $10^{-4}$.

### 5.2 *Papers*

Loehr et al (2013) discussed work that they have done for the Missouri Department of Transportation attempting to develop state-specific resistance factors for Missouri. The guidance offered by AASHTO on which resistance factors to use is very general, prompting this type of effort by individual states in the United States.

The work of Loehr et al (2013) aimed to develop prescriptions for drilled shaft design that would not be overly constraining. The authors worked with epistemic probabilities, which they however do not provide details on. One of the main advantages of LRFD, in these authors' view, is that LRFD enables placing a value on the marginal boring, CPT log or laboratory test, which in turn allows advocacy for a more complete site characterization. The authors illustrate this point with an example of drilled shaft design with site characterization done with different levels of detail. This potential use of LRFD, of facilitating the economic evaluation of site characterization and other decisions affecting design, has also been remarked on by other authors (e.g., Foye and Salgado 2005, Foye et al. 2011a and b).

Katzenbach et al. (2013) review a relatively new soil improvement method that resembles a piled raft. In this method, (usually unreinforced) concrete columns are installed beneath a raft (mat). The mat and columns do not actually touch, being separated by a layer of stone or gravel (Figure 2). The concrete columns, in effect piles, have diameter ranging from 0.25 to 0.80m.

A fundamental difference between design of this foundation solution and that of a piled mat is the presence of negative skin friction, as the layer of gravel pushes down on the soil, which compresses over time between the piles. The authors review French and German prescriptions for design of this type of foundation solution.

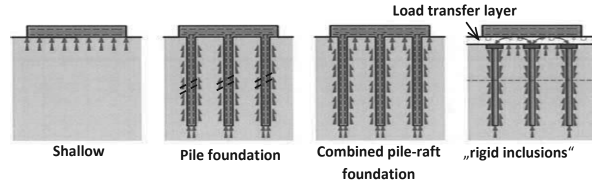


Figure 2 Comparison of common foundations solutions with pile-supported blanket solution (after Katzenback et al. 2013).

### 5.3 *Discussion*

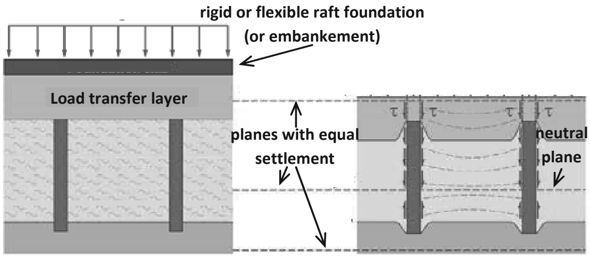


Figure 3 Occurrence of negative skin friction along unreinforced concrete piles caused by load applied by the layer of gravel on top of the column on the soil between the columns (after Katzenback et al. 2013).

## 6 EARTH RETENTION

### 6.1 *General Remarks*

Fundamental work on the probabilistic analysis of retaining walls has tended to focus on mechanically stabilized earth (MSE) walls. This work has resulted in part from the interest sparked by the AASHTO LRFD mandate in North America and the fact that MSE walls are widely used in transportation infrastructure.

While retaining wall limit states in general should be treated similarly to the slope stability limit state (because the pressure on the walls results from shear surfaces that develop in the backfill and that depend very much on spatial variability of shear strength in the backfill), the presence of an interface (the back of the wall) again allows the effects of the spatial variability of soil variables in the backfill to show as variability in contact pressure on the wall. Likewise, reinforcing elements in MSE walls will develop pullout resistance and exert stabilizing action on the backfill through the shear stresses (unit resistances) between them and the soil; variability of these unit resistances are then used in reliability analyses. In the case of MSE walls, instrumentation of reinforcements, including near the wall facing, provide data that can be used for estimation of both earth pressures and unit interface resistance of reinforcing elements (see, for example, Kim and Salgado 2012a,b).

An interesting issue in connection with retaining walls is the choice of limit states that must be checked in design. The traditional, idealized retaining wall limit states are sliding, overturning, bearing capacity failure and general instability. In MSE walls, the additional limit states of pullout and reinforcing element rupture must also be checked. However, as pointed out by Loukidis and Salgado (2012), realistic limit states tend to be a composition of these idealized limit states. Equally interesting, there is a relationship between mobilization of shear strength (and consequently pressures on the wall) and wall movement, and this relationship has implications for the setting up of design situations. Merrifield et al. (2013) discuss aspects of this issue as well, as did Loukidis and Salgado (2012) and Simpson and Driscoll (1998).

### 6.2 *Papers*

Ho et al. (2013) used the design approach proposed in CIRIA Report C580 in the design of a permanent cantilevered, large-diameter, bored-pile wall (Figure 4) for the support of sloping ground bordering a new road, which now exists in front of the wall. This is the first known Hong Kong project in which a permanent retaining structure was designed using the C580 design approach. The wall, which is approximately 110m long, is made of 33 bored piles with diameter equal to 3.0m. Figure 5 shows a cross section of the wall as well as the original and post-construction ground profile.

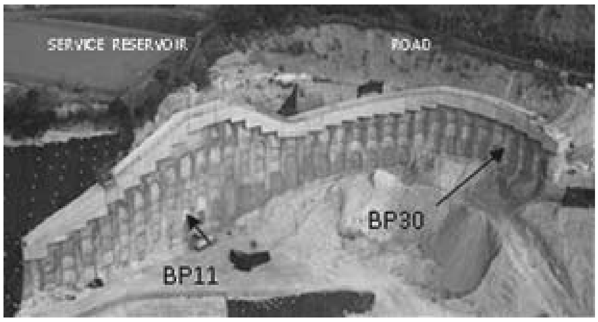


Figure 4 Bored-pile wall in Hong Kong (after Ho et a. 2013).

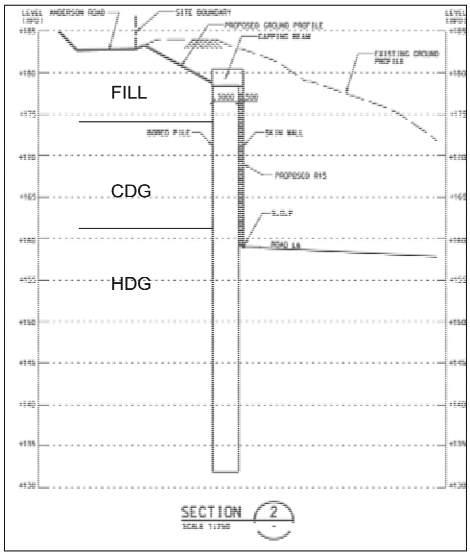


Figure 5 Cross section of bored pile wall in Hong Kong (after Ho et al. 2013).

The ground profile at the site consists essentially of fill and weathered granite. In the original ground profile, the fill had a maximum thickness of 8m and contained loose to medium dense, gravely silty sand or sandy clayey silt, with some rock and concrete fragments as well as domestic waste. Completely decomposed granite (CDG) and highly decomposed granite (HDG) are present below the fill. The granites in the area are commonly fine- to medium-grained and greyish pink to pinkish grey in their fresh state. The thickness of the CDG ranges from 5 m to 10 m with SPT blow counts ranging from 15 to 100. The HDG occurs with SPT blow coutns greater than 100. The maximum depth to moderately to slightly decomposed granites (M/SDG) ranges from 15m to 40m below original ground level. The water level was assumed at the excavation level on the excavation side of the wall and within the the CDG layer on the retained side. In the site charactirization stage, a total of 105 consolidated undrained (CU) triaxial tests, with pore water pressure measurement, were carried out. Soil design parameters for the fill, CDG and HDG were determined based on this testing program.

Ho et al. (2013) used two methods to design the wall. One method is the traditional method used in Hong Kong. The other is the CIRIA C580 method. Use of the C580 design approach netted 10 % overall cost savings over what the wall would have cost if designed using the traditional method. This represented an amount of HK$ 9.6 million saving on this project. This case history illustrates that geotechnical engineering still relies on design methods that may be too conservative, and that savings can be significant when more recent knowledge is put into use.

Merrifield et al. (2013) review the history of and difference in current practice of design and testing of ground anchors in Europe. Ground anchors were first used in rock applications, but found their way into soil retention systems in the 1950s. Ground anchors are routinely used in connection with embedded (cantilevered) walls and may be viewed as enablers of this type of retaining wall, which would otherwise be prohibitively expensive.

Merrifield et al. (2013) discuss the differences between the treatment of anchors in European national codes and provisional (draft) prescriptions in Eurocode (EN-1997:2004). Prescriptions must address the two primary limit states: an ultimate limit state that will prevent pullout of an anchor under the maximum expected load it will be subjected to during its service life and a serviceability limit state related to deflections, which may be stated in terms of loss of load or creep (in fact, an anchor is likely to both move or deform and see a degradation in the load it carries over its service life).

The authors call attention to an important, if not universally recognized, detail concerning definition of limit states for walls: serviceability limit states develop with retaining wall movement that is typically not sufficient to develop active or passive pressures historically assumed to act behind or in front of the wall. As a consequence, serviceability limit states must be defined separately from ultimate limit states. From the point of view of anchor design, anchor resistance must be sufficient to carry loads transferred to it by either serviceability or ultimate limit states, and the draft provisions do account for that as well.

In part because of the empirical basis for anchor design, proof loading has always been part of ground anchor design and installation practice. Again, each country handles testing and proof loads differently, a point that Merrifield et al. (2013) discuss in detail.

## 7 SUMMARY AND CONCLUSIONS

Safety and serviceability are the requirements that any geotechnical design must meet. These requirements are met through risk management, specific design checks and proper construction. The design checks are calculations that show that no limit state is achieved or exceeded. One of the main tasks of the geotechnical designer is to properly identify applicable limit states and then produce a design that keeps the probabilities of their occurrence below threshold levels. Guidance on identification of limit states and specific checks to be done in connection with these limit states are often prescribed by codes. Recent codes, such as the Eurocode and LRFD codes attempt to anchor the design process the notion of an acceptable probability of failure, which depends on the problem and consequences of achievement of a specific limit state. In the absence of specific guidance, definition of what these limiting probability values are sometimes must also be defined by the designer.

The papers submitted to the XVIII ICSMGE illustrate the complexity of probabilistic limit-states based design and encapsulation of it in design codes. Much work still remains for researchers and code development agencies to produce codes and methods of analysis that will enable engineers to produce designs that achieve safety and serviceability in an optimal and economical manner.

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