State of the art report: Analysis and design

Analyse et conception : un état de l'art

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ABSTRACT

This paper is concerned with analysis and design across the breadth of geotechnical engineering. Sections by individual experts relate to Codes and standards, Deep foundations, Embankments and slopes, Underground construction and Seismic design. Each considers recent developments, including economy and sustainability. Recent geotechnical codes and standards from Europe, North America and Japan are compared. All adopt forms of limit state design and address familiar geotechnical problems of uncertain ground behaviour and complex interaction of loads with frictional materials. Some topics which are still under debate are discussed. Design of deep foundations is a blend of empiricism and theory. Developments including drilled and grouted piles in rock and offshore driven piles are presented. Topics considered include assessment of ultimate capacity and the environmental benefits of pile re-use and piles as heat transfer elements. Design of embankments and slopes depends on empirical rules and observation, as well as theory. In Japan, tolerance to displacements following earthquakes is a particular consideration. Embankments used as transport corridors or as building platforms are discussed. Failures involving uncontrolled fills are particularly noted. For construction underground, understanding of the fundamental physics is imperative, along with comprehensive process of checking during both design and construction. Greater use of cement replacements would aid sustainability. The final section of the paper considers performance-based design of foundations against two seismic hazards: emergence of a rupture underneath a structure, and bearing capacity mechanisms for slender structures on shallow foundations.

RÉSUMÉ

L'article étudie l'analyse et la conception d'ouvrages géotechniques variés. Les différentes sections abordent les codes et standards, les fondations profondes, les pentes et talus, les constructions souterraines, puis le génie parasismique. Les développements récents, notamment en matière d'économie et de développement durable, sont présentés. Les codes et standards utilisés actuellement en Europe, aux Etats-Unis, et au Japon sont comparés. Tous adoptent des formes de conception aux états limites et abordent les problèmes courants causés par les incertitudes sur le comportement du sol, et par les interactions structures-milieux frictionnels. Certains sujets encore en débat sont abordés. La conception de fondations profondes mélange empirisme et théorie. Plusieurs développements sont présentés, tels les pieux forés-cimentés dans les roches et les pieux battus en mer. Estimation de la capacité portante ultime, intérêt environnemental de la réutilisation de pieux, et utilisation des pieux pour transférer la chaleur sont, entre autres, considérés. La conception des pentes et talus repose sur des lois empiriques, l'observation, et la théorie. Au Japon, la tolérance aux déplacements dus aux séismes est d'une importance particulière. L'utilisation des talus comme couloirs de transport, ou comme plateformes pour les constructions, est abordée. Les ruptures de remblais non contrôlés sont aussi étudiées. Les constructions souterraines exigent une bonne compréhension de la physique fondamentale et la vérification des phases de conception et de construction. Une utilisation plus fréquente de substituts au ciment serait plus environnementale. La dernière section concerne la conception des fondations contre deux phénomènes liés aux séismes: apparition de discontinuités sous la structure, et modes de ruptures par perte de capacité portante des fondations superficielles de constructions élancées.

Keywords design, analysis, codes, standards, piling, tunnelling, slopes, embankments, seismic

1 INTRODUCTION

The president of the ISSMGE, Professor Pedro Seco e Pinto, invited an international group of experts to prepare this State of the Art Report on Analysis and Design in geotechnical engineering. It was agreed that five topics would be included in the report: Codes and standards, Deep foundations, Slopes and embankments, Underground construction and Seismic design.

Each author was asked to provide a brief review of the fundamentals of technical understanding of most importance to their topic and to comment in particular on recent developments, especially since previous state-of-the-art papers on the topic. Comments are provided on what goes wrong, and emphasis is given to questions of economy and sustainability.

Design is decision. In the context of this paper, design is the complete process or sequence of decisions which determine what is actually built. The decisions may be taken by clients, consultants, contractors, site workers or others – all are part of the design process. Analysis is one part of design, involving calculations carried out by engineers, with or without the aid of computers. The main thrust of the contributions relates to design aspects. References are made to supporting analysis, and it is assumed that advanced techniques such as finite element computations will be available and used when appropriate.

2 CODES AND STANDARDS

2.1 *Introduction*

Codes and standards aim to set out the process and procedures of design, or at least to identify the basic ingredients and limits of acceptable good practice. Figure 2.1 shows the functions performed by codes in many projects, providing the communication between society, clients, data and analysis in the process of design.

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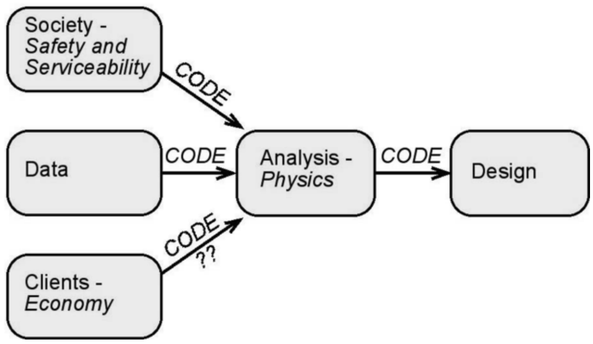


Figure 2.1: Communication facilitated by codes

In this paper, the term "code" will be used in a broad sense. It includes "design standards" such as the Eurocodes, "design codes" such as Japanese codes, and "design specifications" such as the AASHTO LRDF Bridge Design Specifications. The older style of British "code of practice", which often included a more discursive account of the geotechnical design process rather than definitive rules, is within the scope of this discussion, but specifications for construction are outside the scope.

Part of the function of a code is to provide a link between analysis and design, showing how analysis can be used as a tool in the design process. Codes may provide limits on the analysis methods to be used, deliberate or unintended, and generally provide safety factors or margins, establishing gaps between what is analysed, failure states and what is most likely to occur in practice.

Society requires safety and serviceability. Codes provide a link between the activities of the designer and the requirements of society. They are the point at which society's qualitative requirements that structures should be at least as reliable as they traditionally have been are interpreted in the forms of procedures and numbers. Society, or at least clients, also requires economy, to be balanced with safety and serviceability. Although codes have often promoted safety and serviceability with less regard for economy, they also have a responsibility here, especially in a world in which limitations of resources are increasingly recognised.

Perhaps the greatest challenge for drafters of geotechnical codes is to find the best balance between fixed rules and personal expertise, both of which are essential to successful design. Geotechnical engineering relies heavily on the knowledge and judgement of individual designers. Inevitably this is somewhat subjective, depending on the training and experience of each individual, but equally it is indispensably valuable, especially when shared in discussion with others; codes must allow and encourage this. For both individuals and the profession as a whole, past mistakes are a major source of learning, and both must aim to avoid repetition of these. Codes are an important part of the profession's corporate memory, helping to assure society that past mistakes will not be repeated. Equally, though, modern codes are aiming to pull professional practice forward, closer to the best available "state of the art".

In the following sections, approaches currently taken by codes to geotechnical analysis and design will be reviewed and compared. Particular emphasis will be given to European, North American and Japanese developments, noting both the broad areas of agreement and particular points of debate within and between these communities. Specifically, the main documents included in the review are the Eurocodes, particularly Eurocode 7 – EN1997-1 (2004), Geotechnical design; the AASHTO LRFD Bridge Design Specifications (2008); and Japanese Geocode 21: Principles for Foundation Designs Grounded on a Performance-based Design Concept (Japanese Geotechnical Society 2006).

The Eurocodes use the term "actions" in place of the term "loads" used in American and older British codes. English translations of Japanese documents use both terms. In this paper, the term "loads" will generally be adopted, except in direct quotations.

Clause numbers from codes are shown in brackets thus: { }.

2.2 *Limit states and working states*

Investigators of failures frequently report that the main cause was some onerous condition which had not been considered in the design. In contrast, it is often noted that failures were not caused by normal or even somewhat excessive statistical variations of parameters which had been properly reviewed and for which standard margins of safety had been allowed. It appears to be important, therefore, that designers' attention is drawn towards a reasonable range of extreme situations against which to check each design. This is the underlying philosophy of limit state design.

Limit states are "states beyond which the structure no longer satisfies the relevant design criteria". That is, the structure, or part of it, is on the point of "going wrong" – failing in a sense that would be understood by a client, user or non-technical observer. All the codes place the limit state on the safe side of the failure, ie when the structure is almost failing but has not yet failed. Limit states are classified into various types, used selectively in the codes.

In the Eurocodes, *Ultimate limit states* (ULS) are "associated with collapse, or with other similar forms of structural failure". They generally relate to danger or severe economic loss. *Serviceability limit states* (SLS) "correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met". These are unwanted but more manageable if they occur, allowing the structure to remain in service if necessary repairs are carried out. They may be subdivided into irreversible SLS (eg cracking) where the consequences of exceedance remain after the disturbing action is removed and reversible SLS (eg unpleasant vibration) where the consequences do not remain. The Eurocodes also consider accidental design situations, though these are regarded as potential causes of limit states, rather than limit states in themselves.

The Japanese Geocode 21 emphasises that the ULS is still on the safe side of collapse: "the structure may have sustained considerable damage, but not to the extent that the structure has reached failure, become unstable, collapsed, or to the extent that would result in serious injury or the loss of life." Similarly, at SLS there is to be no effect on durability and "regular use of the structure is possible, without repairs." This code also has a *Reparability limit state* "in which damage to the structure has occurred and may have affected the durability of the structure. However, regular use of the structure is possible to a limited extent and there are reasonable prospects for full functionality of the structure if economically-feasible repairs are performed. This limit state can be interpreted as the state in which the majority of the value of the structure has been preserved. In addition, the reparability limit state sometimes implies a state in which marginal use of the structure is possible for rescue operations immediately following an extraordinary event such as a large earthquake."

The AASHTO Bridge Code has *Strength limit states* "relating to strength and stability during the design life". This is a more technical definition than the Eurocodes' ULS, but from most points of view it serves a similar purpose. The same code has *Service limit states* "relating to stress, deformation, and cracking under regular operating conditions". AASHTO also have *Extreme event limit states* "relating to events such as earthquakes, ice load, and vehicle and vessel collision, with return periods in excess of the design life of the bridge." These may be compared with the Eurocodes' *accidental design situations* noted above.

Offshore codes such as Det Norske Veritas (1993) include other "limit states" such as *progressive collapse limit state*, *fatigue limit state*, though these may be considered more descriptive of the way the limit state is reached than of the limit state itself. The *survivability limit state*, however, generally does describe a particular state of the structure.

In *Limit State Design*, attention is directed to states at or close to failure, which it is hoped will not occur in practice. The alternative is *Working State Design* in which attention is directed to the expected, desired state in which the structure is performing successfully under its expected loading. Its safety is then usually checked by requiring that the degree to which material strengths or ground resistances are mobilised is limited.

Limit state design codes usually require that at least two limit states be checked. One of them is normally a serviceability limit state, for which unfactored parameter values are usually used in calculations. This is the nearest approach to an analysis of the expected performance of the structure, but it is more pessimistic than is really expected, as discussed further below. The other limit states involve more severe situations, often derived by applying partial factors to selected parameters representing loads, ground or structural material strengths, ground or structural resistances, and possibly other parameters including water pressure.

Despite some differences of emphasis, or of drafting, the developers of the modern codes agree that a limit state approach, concentrating on failure, or near failure, is to be preferred (see, for example, Becker, 1996). The present authors agree with this conclusion, adding the following observations:

1. Limit state design implies that the states studied should not occur. For ULS, their probability is to be extremely low, so calculations are directed to a virtually non-existent state. This may confuse designers who more readily imagine states which they consider fairly likely to exist. In this respect, working state design is easier to understand.
2. It was noted above that actual failures are often caused by the occurrence of situations that had not been properly foreseen during the design process. Some of these might be accepted as virtually unforeseeable, while others are errors of process on the part of the designer, such as misjudgement of soil properties or failure to check a critical mechanism. It is essential, therefore, that codes continue to emphasise good geotechnical practice and process, avoiding too great a concentration or reliance on factors of safety.
3. Calculation errors are discovered very often during investigations of failures. Complexity in codes seems likely to increase the number of errors, so it is important, as a safety issue, that code provisions are kept as simple as possible.
4. A few of the calculation errors prove to be critical, but it seems likely that many others are covered by the margins of safety traditionally in use. Human error is therefore a significant uncertainty that must be included in any rational attempt to determine factors of safety. Failing this, overall factors of safety can only be reduced with the utmost caution.
5. The aim, however, should be to reduce factors of safety, and certainly not to increase them, in general. Unnecessarily large factors of safety lead to increase in cost, and wastage of materials and energy.

2.3 *Assessment of parameter values – characteristic values*

Prescribed values for partial factors provided by codes have little meaning unless it is clear how the parameter values are initially to be selected before being factored. These unfactored parameters are referred to as *characteristic values* of loads, strengths or resistances in Eurocodes and Geocode 21, and as *nominal values* in AASHTO. In effect, modern codes provide safety margins by a combination of two features: somewhat cautious characteristic values, determined by the designer in the case of soil properties, and the application of partial factors, prescribed by the codes on behalf of society. Usually, "cautious" values of strength are lower than the most probable, but for some limit states values on the high side are more critical.

In the case of loading, the values themselves are largely prescribed by the loading codes. For manufactured materials, characteristic values of strength are derived as a fractile of the test results of a closely specified type of test, typically 2 standard deviations from the mean, giving a frequency of about 2.3% in the tests. It is commonly the case that code drafters are more knowledgeable about both loading and structural material properties than are designers. For ground material the situation is more complex, however, for several reasons: (a) ground materials cover a wide range of types, some more easily sampled than others, so a range of testing approaches is unavoidably adopted; (b) in most, but not all cases, the parameter relevant to the design is close to a mean value, averaged over a large zone or surface within the ground, rather than to an extreme derived from testing small samples; (c) the uncertainty and variability of the ground on a particular site is often much better known to the designer than it could be to the code drafter; (d) the process of construction sometimes changes the ground properties; (e) data and observations obtained from similar projects may be highly relevant to selection of appropriate parameter values; (f) full scale testing is quite often incorporated into the design process, particularly for piles and ground anchors.

This problem is not new: in past practice it was usually unclear how conservative the code drafters assumed the values of parameters to be. An early attempt at providing more guidance was represented by CIRIA Report 104 (Padfield and Mair 1984) which asked users to assess either "moderately conservative" or "worst credible" values for soil strength parameters and provided differing factors for these two cases. The same approach is taken in the more recent replacement of this report, CIRIA Report C580 (Gaba et al 2003).

The developers of various geotechnical codes have taken different approaches to this problem. After very lengthy debates in Eurocode 7, the basic definition of characteristic soil parameters was agreed as: "a cautious estimate of the value affecting the occurrence of the limit state". Additional text makes it clear that this estimate is to include allowance for the extent of ground involved in the limit state, and hence average effects over that extent, and the effect of construction activities on ground properties. In making this estimate, input from laboratory and field tests, and also well-established experience are required, but the type of testing is not restricted and a distinction between situations in which spatially averaged values or extreme values govern is encouraged. It is suggested that if statistical approaches are applied to the data "the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%". Thus the emphasis throughout is on engineering assessment of the values actually governing behaviour in the ground, which might differ from those measured in tests, with a requirement that the assessment be "cautious", not a best estimate of the most likely value, especially if there is considerable uncertainty. It is implied that "cautious" means that the chance that values worse than the selected value will actually occur in such a way as to be the cause of a real problem (a limit state) is about 5%. The expertise of the engineer is by no means relegated, but is incorporated into the assessment of the characteristic values. The present authors would argue that this is no different from previous good practice, and essentially the same as the "moderately conservative" values of CIRIA Report 104; engineers have generally aimed for a mean value, but have been slightly cautious, especially when the degree of uncertainty was significant.

It is important to recognise that in a typical situation involving large amounts of ground a "probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%" is very different from a 5% fractile of test results. Schneider (1997) discussed situations where the zone of ground affected is large enough that its behaviour will be close to the overall average of the soil considered. He argued that a "cautious estimate" with a 5% chance of a worse overall value would lie about 0.5 standard deviations from the mean of relevant measured values, adjusted as necessary for sampling disturbance and construction effects. The marked difference between these two assessments can be seen in Figure 2.2.

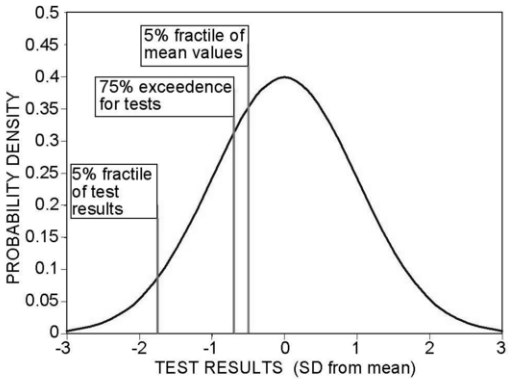


Figure 2.2: Alternative derivations of "characteristic values"

A similar proposal was made by Dahlberg and Ronold (1993) for design of offshore foundations and recommended by Becker (1996) for more general use. This involves the use of a "conservatively assessed mean" (CAM) as the characteristic value, also about 0.5 standard deviations from the mean of the test results. These authors state that for a normal distribution 75% of the measured values would be expected to exceed this value. (More accurately, this requires an offset of 0.69 standard deviations from the mean, for a normal distribution, as shown in Figure 2.2). Foye et al (2006b) take up the same idea proposing to use a CAM with 80% exceedance, equivalent to 0.84 standard deviations below the mean of the test results for a normal distribution. These proposals are made in the development of North American practice, though at present the AASHTO Specifications do not use the term "characteristic" but refer less specifically to nominal values related to "permissible stresses, deformations, or specified strength of materials".

Japanese Geocode 21 {2.4.3} defines characteristic values of geotechnical parameters as "the representative values that have been cautiously estimated as the most appropriate values for the foundation-ground models in order to predict the limit states checked in the design"; this is apparently close to the Eurocode definition. It continues: "The characteristic value of a geotechnical parameter is principally thought of as the average (expected value) of the derived values", but it adds qualifications to this statement that diverge away from a simple statistical average, and shows the same in a mathematical formula. The terms "cautiously estimated" and "expected" are potentially in conflict, but the overall impression given is that the characteristic value may be somewhat more conservative than the statistical mean. However, commenting on Geocode 21, Honjo et al (2005) particularly emphasise that "the characteristic value is defined as a mean value of a geotechnical parameter".

Thus there seems to be approximate agreement between the Eurocodes and the North American proposals about the level of conservatism most usefully associated with the characteristic value; the Japanese position might be slightly different. The recommendations of Becker and of Foye et al, intended for North American practice, encourage the adoption of a limited range of specific tests, CPT and SPT, with the characteristic value derived directly from the test results (including modifications for stress level and possible removal of rogue values). This would largely remove judgement from the process, although considerable geological judgement remains in zoning the site, both in plan and in level. In contrast, the European approach incorporates the designer's expertise into the assessment of characteristic values. It allows assimilation of results from a range of test types and sources, and encourages engineers to consider the relevance of the test results to the way the ground will respond at a specific limit state, including effects of construction on its behaviour. Eurocode 7 also notes the need to be aware that extreme values of strength may be the critical ones if a failure mode could exploit them.

An alternative approach is also mentioned by Becker (1996) and has been used in Swedish practice (Boverket, 1995). In this approach, the "Characteristic value" is taken to be a simple mean value, or best estimate of the actual value. The value of applied partial factors is varied in a prescribed manner related to the assessed reliability and variability of the data from which the mean was derived, and its likely ductility. The author understands that Swedish codes are changing towards a definition which includes some of the uncertainty in the choice of characteristic value, and is therefore closer to the Eurocode definition.

With relatively small differences in approach, all these documents share the aim of obtaining the most useful, rational and transparent combination of objective test results with engineering knowledge and experience, necessarily subjective to some extent. A purely objective process would disregard large amounts of human knowledge which are essential to geotechnical design.

2.4 *Can the various limit states really be treated separately?*

In principle, limit state design requires separate consideration and analysis of various different limit states, for example, ultimate (or strength) and serviceability limit states. Separation makes it easier to specify the requirements of design for each limit state, and many authors, eg Becker (1996), have argued that this is essential for the "rational" derivation of values for partial factors, ie on a probabilistic basis. The split between ultimate and serviceability has encouraged development of testing methods and analytical tools for serviceability calculations, generally aiming to provide ground stiffnesses for calculation of displacements. The more detailed prescriptions of the codes have generally been dominantly ULS, partly because SLS criteria are the province of the client rather than being requirements of society.

Parameter values and related calculation models giving reliable predictions of displacement are often very difficult to obtain. In contrast, those for mechanisms of plastic failure are generally easier and more reliable; for example, the angle of friction of a soil is much easier to obtain than its deformation properties. Because of this, Eurocode 7 also allows SLS to be covered by the strength-based (plastic mechanism) calculations in some cases, as shown by the following extracts.

{2.4.1(4)P} If no reliable calculation model is available for a specific limit state, analysis of another limit state shall be carried out using factors to ensure that exceeding the specific limit state considered is sufficiently improbable. Alternatively, design by prescriptive measures, experimental models and load tests, or the observational method, shall be performed.

{2.4.8(4)} It may be verified that a sufficiently low fraction of the ground strength is mobilised to keep deformations within the required serviceability limits, provided this simplified approach is restricted to design situations where:

- a value of the deformation is not required to check the serviceability limit state;
- established comparable experience exists with similar ground, structures and application method.

{6.6.2(16) [for spread foundations]} For conventional structures founded on clays, the ratio of the bearing capacity of the ground, at its initial undrained shear strength, to the applied serviceability loading should be calculated (see 2.4.8(4)). If this ratio is less than 3, calculations of settlements should always be undertaken. If the ratio is less than 2, the calculations should take account of non-linear stiffness effects in the ground.

{7.6.4.1(1)NOTE [for pile design]} For piles bearing in medium-to-dense soils and for tension piles, the safety requirements for the ultimate limit state design are normally sufficient to prevent a serviceability limit state in the supported structure.

The first of these extracts refers to limit states in general. The second and third occur in sections on SLS, allowing strength calculations which may be different from those required for ULS to be used to demonstrate limited settlement. It will normally be the case that this SLS condition is, by inspection, more severe than the ULS condition which would use the same type of calculation, so one calculation checks both SLS and ULS. The fourth extract, referring to piles, specifically states that the ULS calculations will normally cover SLS also; again, one calculation covers both. In the European system, actual values for partial factors are set by each nation, so each must ensure that the values set comply with this note; the application of this in developing the UK National Annex for EC7 (BSI 2007) is discussed by Bond and Simpson (2009).

If the values of partial factors were derived by probabilistic analysis, it would be confusing to mix SLS and ULS, as noted by Becker (1996). However, in practice the values adopted in codes to date are mainly chosen to reproduce existing experience of successful behaviour. Since this implies success both in ULS and SLS, it is clear that the factors being chosen are adequate in most cases to provide for both, and it may be very difficult to determine which of the two limit states actually governs the factor values. Factors based on past successes apparently cover all significant variables, to a degree which is generally adequate. Besides uncertainly of loads and material strengths, this includes calculation models, geometric uncertainties and human errors.

In the opinion of the authors, statistical studies are useful to code drafters in helping to allocate partial factors in appropriate proportions between the parameters. However, when the overall level of safety derived by modern codes is fixed by calibration with past successful experience, actual magnitudes may well cover more than one limit state and inevitably allow for a greater range of uncertainties than the statistical variation of the basic parameters. A lack of theoretical purity in approach is probably of little concern to designers, provided codes are clear about what limit states are covered by each of their requirements, and in particular when it is necessary to carry out specific calculations of deformation and displacement.

2.5 *Review of the alternative approaches*

2.5.1 *Working state design*

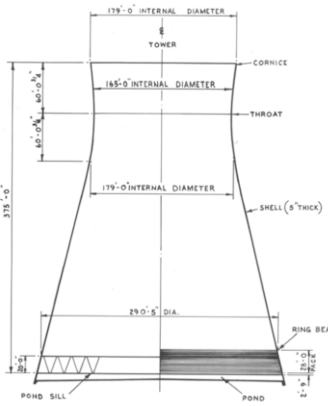
As noted above, in *Working state design* attention is directed to the expected, desired state in which the structure is performing successfully under its expected loading. Its safety is then usually checked by requiring that the degree to which material strengths or ground resistances are mobilised is limited. This means that "permissible stresses" are not exceeded, so the approach is also called *Permissible stress design or Working stress design (WSD).*

Working state design has the advantage that the designer is asked to consider states which are easier to imagine because they are likely to occur. Its weaknesses became apparent in structural design in situations where unfavourable and favourable loads tend to cancel in the expected state, leaving only small working stresses, but where a slight increase in the unfavourable loads could lead to proportionately very much bigger increases of stress.

In November 1965, during severe wind conditions, three of the eight cooling tower shells collapsed at the Ferrybridge C Power Station in Yorkshire, UK (Figure 2.3). The Committee of Inquiry into the collapse reported that wind conditions were "very considerably lower than the probable maximum values" and that the structural analysis had been carried out correctly. They reported: "The important membrane stresses in the shells of the towers are the resultant of compressive stresses due to dead weight and tensile stresses due to wind uplift: the resultant tensile stress, being the difference between two large quantities, is sensitive to small variations in the wind loadings." This event was critical in the rejection of working state design and development of load factoring, indicating the importance of considering each independent load separately.

Ferrybridge was not a geotechnical failure, but all the loads in any structure have eventually to be brought down to the ground, so the same thinking applies to foundation and geotechnical designs. Schuppener et al (2009) discuss the examples shown in Figure 2.4. In each case, the forces transmitted to the ground may differ appreciably from the most likely working state due to relatively small changes in the structure and its loading. In Figure 2.4a, small changes in loading change the forces in the piles from tension to compression. In Figure 2.4b, small changes determine whether an anchor is needed or not at point A.

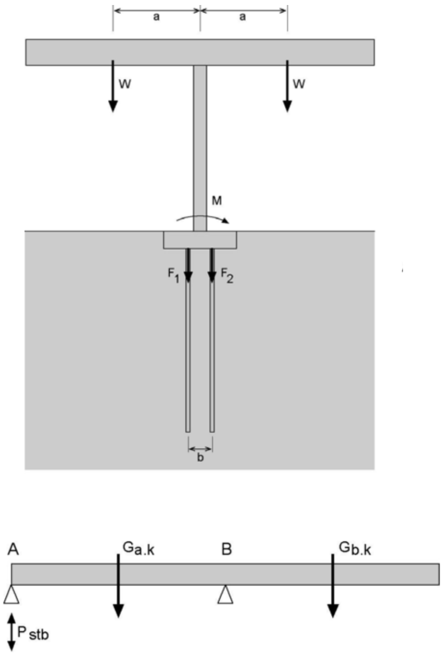
To avoid problems of this type, the drafters of modern codes have taken the view, rightly in the opinion of the present authors, that designers should be required to consider states in which relatively extreme values of parameters take the structure close to failure.



(a)

(b)

Figure 2.3: Collapse of Ferrybridge cooling towers



(a)

(b)

Figure 2.4: Examples of balanced loads (after Schuppener et al 2009)

2.5.2 *Introduction of partial factors*

For ultimate or strength limit states, the requirements can generally be resolved into a condition of the form $E_d \leq R_d$. That is, the design effect of loads must not exceed the design resistance, the term "design" implying that all necessary factors are already incorporated. This condition may be checked at many different points in a structure, typically for example as a thrust, tension, shear force or bending moment in a structural member, or as a bearing pressure, disturbing moment or lateral resistance in a geotechnical calculation. The term E_d is formed by the combination of separate loads with their own factors and the term R_d is calculated from characteristic values of ground strength using theoretical or empirical relationships of varying provenance and reliability. Significant differences of opinion exist in deciding where in the process of deriving E_d and R_d the factors should be applied.

Eurocode 7 effectively allows a very general equation:

$$\gamma_E\, E\{\gamma_F\, F_{rep};\, X_k/\gamma_M;\, a_d\} = E_d \leq R_d = R\{\gamma_F\, F_{rep};\, X_k/\gamma_M;\, a_d\}/\gamma_R \qquad (2.1)$$

in which the main variables are F_{rep}, X_k and a_d – representative loads (or "actions"), characteristic ground strengths and design geometrical parameters (representative actions are derived from characteristic actions using load combination factors, outside the scope of this paper). The functions E{ } and R{ } represent the derivation of the load effect from the individual loads and of the corresponding resistance from the ground strengths and other parameters. The factors are γ_E, γ_F, γ_M and γ_R on load effects, individual loads, material strengths and resistances, respectively, some of which may be set to unity. The material strength term X_k is included in the load effect side of the condition because ground strength may affect load effects such as lateral earth pressures; similarly the load term F_{rep} is included on the strength side because in frictional soils loads may affect strength – for example normal loads enhancing sliding resistance. In North American usage, the reduction factor ϕ_R is used in place of $1/\gamma_R$.

Between the various groups working on code development, opinions have diverged about which of the factors γ can most usefully be set to unity, their effect being incorporated into other factors. All developers aim to provide a system which is robust and reasonably simple to apply. In the author's experience, the greatest complexities and lengthy debates arise in design practice when the system proposed leads to obviously unreasonable results; sometimes this occurs as a result of over-enthusiasm for simplicity.

2.5.3 *Factors applied to material strength or resistances*

On the resistance side, the main debate is whether to apply factors γ_M to material strength (c_u, c', $\tan\phi'$) or factors γ_R or ϕ to derived resistances such as bearing capacity or lateral resistance. Becker (1996) put the case for resistance factors as follows:

"Although the factored strength approach may be considered as being more elegant and sophisticated, the factored overall resistance approach has a significant advantage over the factored strength approach in that the derived resistance factor reflects not only uncertainty in strength but also uncertainties associated with the analytical models, site conditions, construction tolerances, and failure mechanisms. The factored strength approach alone does not capture all sources of uncertainty in the calculation of resistance. There is a need to also take into account the uncertainties stated above and others such as the development of excess pore-water pressure and stress-strain behaviour. The factored strength approach also does not capture the true mechanism of failure when failure is influenced by nonlinear soil behaviour. … The factored resistance approach is similar to WSD and may be viewed as a logical extension to WSD. Therefore, it would be familiar to and, hence, better received by geotechnical engineers, which would allow for a smoother transition from WSD to LSD for foundation design."

In Eurocode 7, both possibilities are allowed and the choice is left to individual nations. There has been no general agreement as to which approach is preferable or, indeed, whether either approach is best in all circumstances. The main argument voiced in favour of the resistance factor approach has been the second main point made above by Becker, that this more readily reflects existing practice based on single factors of safety. However, some European countries diverged from that system many years ago; examples include CIRIA Report 104 (Padfied and Mair 1984), the Danish code (Dansk Ingeniorforening 1984), Norwegian codes (Det Norske Veritas 1993), and the British retaining walls code (BS8002 1994). It would be difficult to imagine a single set of factors which could adequately take account of "uncertainties associated with the analytical models, site conditions, construction tolerances, failure mechanisms, … excess pore-water pressure and stress-strain behaviour". The Eurocode approach is generally not specific about which analytical models are to be used but requires that they should be "accurate or erring on the side of safety"; where necessary, an additional resistance factor (or "model factor") may be introduced to ensure this. The code requires other uncertainties such as construction tolerances to be considered specifically if they are significant and similarly requires either that extreme pore pressures are considered or that load factors are applied directly to them. In all systems, minor variations in these secondary variables are implicitly covered by the factors on the leading variables.

Simpson (2007) considered some of the advantages of a material strength approach in geotechnical design, in particular:

- It facilitates consistent analysis of combined problems, which are very common, involving, for example, a slope, loaded by a structure, supported by a retaining wall, itself supported by anchors and foundations. An example is shown in Figure 2.5.
- Because the strength of soil is derived from friction, non-linear, or disproportionate relationships between soil parameters and resistances are common. In these circumstances, it is important to check designs with factors of safety applied to the basic strength parameters of the soil, as explained fully in EN1990 {6.3.2}. Such non-linear

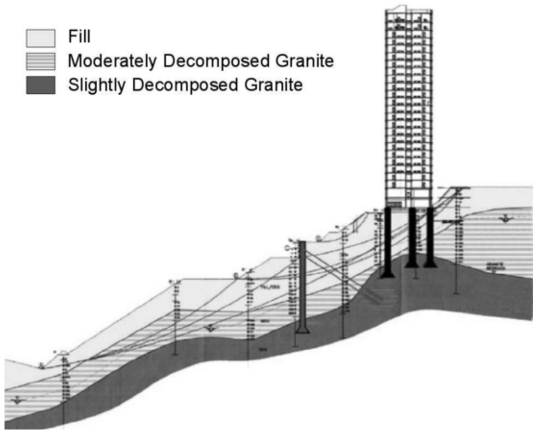


Figure 2.5: Combined geotechnical and structural design situation

relationships occur, for example, in the derivation of bearing capacity or of passive resistance from angle of shearing resistance, as illustrated in Figure 2.6. (K_p is plotted for $\delta/\phi=2/3$.)

- It can be used readily with both simple hand calculations and more complex finite element calculations. Introduction of resistance factors into finite element computations, which essentially require overall equilibrium, has proved to be difficult. In the authors' view, demonstration of equilibrium of complete systems is fundamental to good design practice.

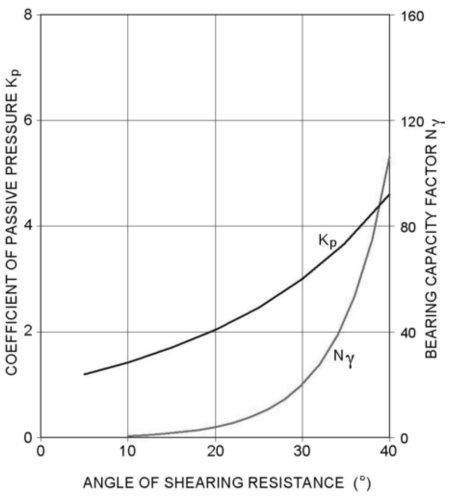


Figure 2.6: Non-linear relationships between material property and resistance.

Foye et al (2006 a,b) report studies intended to derive values for resistance factors ϕ_R for design of spread foundations. Their work is based on statistical studies of the variability of material strengths, as measured by CPT and SPT, and does not incorporate explicitly the other variables listed by Becker (1996). Indeed, these would be very difficult to include since statistical studies of the behaviour of spread foundations in practice are not available. Starting from an assessment of the coefficient of variation of angle of shearing resistance, φ', they aim to achieve constant reliability for bearing capacity and conclude that ideally the factor ϕ_R on bearing capacity should vary as a function of φ'. Figure 2.7 shows how their recommended value of ϕ_R varies with "nominal" (equivalent to "characteristic") value of φ' derived from CPT results. The graph is plotted for a live/dead load ratio of 0.5, and four curves are plotted for four values of reliability index β that they consider to be relevant, with preference for β=3.0. The figure also shows the equivalent value of ϕ_R provided by a constant factor of 1.25 applied to tanφ', a typical value of γ_φ used in EC7; adjustments have been made to allow for the differences between load factors in the two approaches. (The plot for DA2 will be discussed later.) It can be seen that the constant γ_φ provides an equivalent ϕ_R having a similar variation with φ' to that proposed by Foye et al, and corresponds, according to their analysis, to a reliability index β of between 2 and 2.5. The precise value of β is dependent on the way the characteristic or nominal φ' is derived; Foye et al show different values when it is derived from SPTs because greater uncertainty is involved.

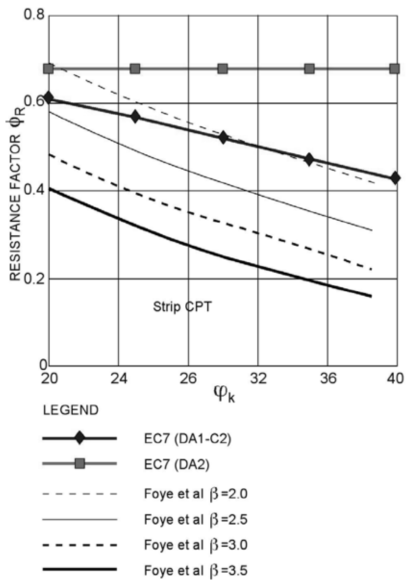


Figure 2.7: Relation of ϕ_R to φ recommended by Foye et al (2006b)

The implications of this comparison for ULS can be seen in Figure 2.8, which shows results obtained by Foye et al (2006b) of the width of strip footings as a function of nominal angle of shearing resistance φ', obtained from interpretation of CPT results. Also shown are calculations following EC7 DA1, plotted against characteristic angle of shearing resistance. IN both cases, the footing with B has been normalised in relation to the soil weight density γ and the total design load ΣF_d used by Foye et al. They use a formula for the bearing capacity factor N_γ taken from Salgado et al (2004) which is more conservative than the EC7 formula, so calculations using the EC7 partial factors are shown for both bearing capacity formulae in Figure 2.8. As might be anticipated from Figure 2.7, the two sets of results are similar EC7 results are similar, with the EC7 results somewhat less conservative.

An interesting feature of this work is that Foye et al based their analysis on an assessed coefficient of variation of angle of shearing resistance, which would most readily be represented by a factor on tanφ' (or φ' - it makes little difference), rather than by a ϕ_R. The same implication was made in previous work by Burland et al (1981) who developed a new method for representing factor of safety in design of embedded retaining walls. Again, they justified it by showing that it was equivalent to a constant factor on soil strength, implying indirectly that factoring soil strength is a sound way to proceed. It seems that geotechnical engineers intuitively expect that a constant factor on tanφ' is a good way to give a reasonably constant level of safety. This may reflect the point noted above that when strength is derived from friction, non-linear, or disproportionate relationships between soil parameters and resistances are of concern.

One problem of a material strength approach is that some calculation methods derive bearing resistance, for example, directly from results of in situ tests, without the use of a

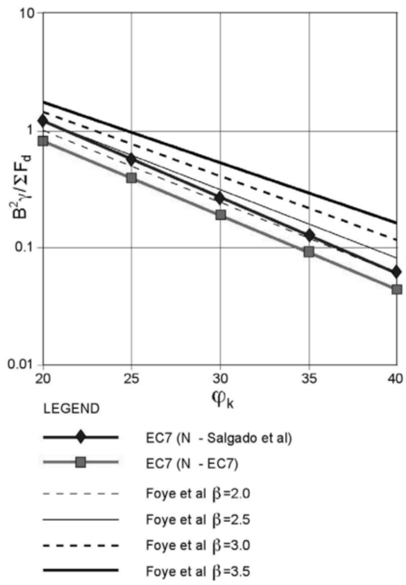


Figure 2.8: Footing dimensions according to Foye et al (2006b) and EC7

parameter of material strength. If it is significant, this difficulty could be overcome by having a resistance factor which varies as a function of the test result, much as suggested by Foye et al. The special case of pile design is considered below.

It was noted above that because factor values are inevitably based on experience to a considerable extent it is impossible to distinguish whether they are required by ULS or SLS considerations. Bolton (1993) has argued that in many cases SLS requirements can reasonably be accommodated by checking the degree of mobilisation of material strength (also Osman et al 2004, Osman and Bolton 2006). This is also readily facilitated by factors on soil strength, relieving code drafters, to some extent, of the need to make the difficult distinction between requirements for ULS and SLS.

2.5.4 *Factors applied to loads or load effects*

The collapse of the Ferrybridge cooling towers, noted above, illustrates the danger of combining loads too early in a calculation. Equation 2.1 includes factors on both loads and load effects. Eurocode 7 allows either to be used, but in either case requires different factors on permanent and variable loads. In general, it could be difficult to distinguish the effects of two different loads after they have been combined into a single load effect. The situations shown in Figure 2.4 are again noted as examples.

This issue is particularly critical to the design of spread foundations because calculated bearing capacity is very sensitive to inclination and eccentricity of load. To augment information culled from literature, the author has discussed this with various experts. In the development of Japanese codes, Shirato (2008) notes that factors are generally applied to loads before combination, though the client body for railway structures requires the use of unfactored loading, with all safety factors on resistances. For the Canadian Highway Bridge Design Code, Green (2008) notes that loads are to be factored before being combined, so the inclination and eccentricity used in the ULS calculation are based on the factored loads. Becker (2008) notes that this question has been debated in North America, without unanimous agreement. The AASHTO LRFD Bridge Design Specifications (2008) uses factored loads to compute eccentricity and Figure C11.5.5-2 shows that earth pressure forces are factored before being combined for calculation of eccentricity, stating "Abutments, piers and retaining structures and their foundations and other supporting elements shall be proportioned for all applicable load combinations" The Specifications state "Bearing resistance shall be investigated at the strength limit state using factored loads" {11.6.3.2}. Despite this, for the calculation of load inclination unfactored loads are used {eg 10.6.3.1.2a}; an earlier edition gave the explanation "To preserve the angle of inclination of the resultant, unfactored loads are used. When factored loads are used, the failure surface beneath the footing could be different than the one due to the applied loads, and the result may become overly conservative." Modjeski and Masters Inc (2003) provide an example of this for a spread foundation for a retaining wall designed to AASHTO Specifications.

The various National Annexes to EC7 take differing views of this question, as described below.

2.6 *Eurocode 7*

EC7 requires that both ULS and SLS be considered. Most of its text refers to ULS; text referring to SLS was discussed above. For ULS, the main approach is based on use of partial factors, but opinions in Europe differ about where and how these should be applied. This is left to national choice, and the values to be adopted for partial factors may also be varied nationally. Three alternative "Design Approaches" have been developed, combining partial factors in different ways; the factor values proposed in the European document are shown in Figure 2.9. Bond and Harris (2008) have summarised the adoption of the three Design Approaches by the various countries in Europe, as shown in Figure 2.10, for designs other than slope stability. Most countries that have adopted DA2 for other purposes have decided to use DA3 for slope stability.

In Design Approach 1 (DA1), two "combinations" of partial factors are specified, and the design must be shown to accommodate both combinations (Figure 2.9). Essentially, they are used in the same way as load combinations, but the concept is extended to include material strengths and resistances. Partial factors are generally applied to either loads (before combination) or ground strengths (before calculation of resistances), though with some exceptions. In countries that use DA1, the factors on ground materials and strengths are generally set to 1.0 in Combination 1; the factors adopted in the United Kingdom are shown in Figure 2.11. For design of piles and anchors, factors are applied to resistances rather than to material strengths, for two main reasons: (a) the designs often involve load testing, which leads directly to resistances, and (b) because the construction process may change the ground strength, a major uncertainty exists in deriving resistances from the strength of the ground before construction; the resistance factors can be seen as factors on this process (or "model") or on the strength of the ground at the interface with the pile or anchor. There are some situations in which factoring loads at source leads to unreasonable situations, especially in the design of retaining structures. For these, EC7 allows the factors to be applied to the effects of the loads, and this is used where appropriate in "Combination 1" of DA1.

In DA2 (Figure 2.9, partial factors are applied to loads and to ground resistances. In a variant of DA2, DA2*, the equilibrium calculation is carried out using unfactored ("representative") loads, and the factors are applied to derived load effects. It has been found that DA2 and DA2* are unsuitable for slope stability problems, so most countries which have adopted DA2 use DA3 for slope stability.

In DA3, factors are applied to material strength and to loads simultaneously, in contrast to the two-combination approach of DA1 in which they are applied to the two separately and the results compared. DA3 has been adopted mainly for slope stability problems only, though a few countries propose to use it for foundations and retaining walls, with factors quite different from those shown in Figure 2.9 in most cases.

Simpson (2007) has emphasised that in many designs involving slopes, retaining structures and foundations ground and structures have to perform together and if a failure occurred

			Design approach 1										Design approach 2					Design approach 3			
			Combination 1			Combination 2			Combination 2 - piles & anchors				DA2 - Comb 1			DA2 - Slopes		DA3			
			A1	M1	R1	A2	M2	R1	A2	M1 or	M2	R4	A1	M1	R2	A1	M=R2	A1	A2	M2	R3
Actions	Permanent	unfav	1,35										1,35			1,35		1,35			
		fav																			
	Variable	unfav	1,5			1,3			1,3				1,5			1,5		1,5	1,3		
Soil	tan ϕ'						1,25				1,25							Structura	Geotech	1,25	
	Effective cohesion						1,25				1,25							actions	actions	1,25	
	Undrained strength						1,4				1,4									1,4	
	Unconfined strength						1,4				1,4									1,4	
	Weight density																				
Spread footings	Bearing														1,4						
	Sliding														1,1						
Driven piles	Base											1,3			1,1						
	Shaft (compression)											1,3			1,1						
	Total/combined											1,3			1,1						
	Shaft in tension				1,25							1,6			1,15						1,1
Bored piles																					
	Total/combined				1,15							1,5			1,1						
	Shaft in tension				1,25							1,6			1,15						1,1
CFA piles	Base				1,1							1,45			1,1						
	Shaft (compression)				1,0							1,3			1,1						
	Total/combined				1,1							1,4			1,1						
	Shaft in tension				1,25							1,6			1,15						1,1
Anchors	Temporary				1,1							1,1			1,1						
	Permanent				1,1							1,1			1,1						
Retaining walls	Bearing capacity														1,4						
	Sliding resistance														1,1						
	Earth resistance														1,4						
Slopes	Earth resistance																1,1				

(shaded box) indicates partial factor = 1.0

Figure 2.9: Factors proposed by CEN for the three Design Approaches

it would involve both of them The main advantages of DA1 were stated to be:

a. Consistent design of complete ground-structure systems with the same two sets of factors.
b. More consistent reliability across a wide range of geotechnical problems including foundations, slopes and retaining structures.
c. Factoring materials mainly at source, better control of safety is obtained in frictional materials which give non-linear relationships to resistances.
d. By factoring loads mainly at source, better control of safety in cases where loads tend to cancel each other.
e. Ready use with finite element methods.

The main reason for adoption of DA2 has usually been stated to be that it is more like previous practice and produces results consistent with previous practice. This has been particularly used in support of DA2*.

Figure 2.10: Adoption of the three Design Approaches by the various countries in Europe (after Bond & Harris 2008).

The author has found that the results of DA1 and DA2 are in most cases fairly close, though DA2* may produce more economic, less safe designs in some cases. As noted above, previous practice varies between countries. For the design problem shown in Figure 2.12, Orr(2005) found similar results for DA1 and DA2. DA1 Combination 2 gives the design length for DA1, which is 4.73m, and the design maximum bending moment is 163 kNm. The design length for DA2 is 4.69m, which is marginally less than the DA1 length, while the maximum design bending moment is 177 kNm, which is slightly greater than DA1. There is no application of DA2* in this example, and DA3 is equivalent of DA1 Combination 2.

It is hoped that some convergence of these approaches will be achieved in the future. The combined used of DA2 and DA3, adopted by some countries, is similar in many ways to the two combinations of DA1, suggesting some progress towards a common approach.

2.7 *Some specific examples*

2.7.1 *Spread footings*

For spread foundations with vertical loading, EC7 Design Approaches 1 and 2 generally give similar results. For example, the factors in Figure 2.9, Orr (2005) calculated widths of foundations within 10% of one another for a typical example. It is found, however, that the sizes of footings computed generally imply high bearing pressures, even for the unfactored case, and it is likely that serviceability checks will dominate the design, leading to bigger footings. Footings designed to ASHHTO (2008) strength limit state would be considerably larger than required by the EC7 ULS. As seen in Figure 2.7, EC7 Design Approach 1 follows the trend proposed in recent studies by Foye et al (2006b), while Design Approach 2 allows for less uncertainty of behaviour at high angles of shear strength.

For combinations of vertical and horizontal loads, however, DA1, DA2 and DA2* may differ appreciably because the load factors are applied at a different point in the process. Figure 2.13 shows an example published by Vogt and Schuppener (2006) which provides an illustration of the effect of factoring loads before or after they are combined. This involves the calculation of the width of a strip footing subject to inclined eccentric loading, based on ULS bearing capacity calculations.

A range of the characteristic value of the horizontal force $H_{Q,k}$ is considered. Figure 2.14 shows that Combination 1 of DA1 requires two separate action combinations to be considered, applying 1.5 to the variable horizontal action, and

			Design approach 1 Combination 1			Combination 2			Combination 2 - piles & anchors				
			A1	M1	R1	A2	M2	R1	A2	M1 or	M2	R4	EC7 values
Actions	Permanent	unfav	1,35										
		fav											
	Variable	unfav	1,5			1,3			1,3				
Soil	tan φ'						1,25				1,25		
	Effective cohesion						1,25				1,25		
	Undrained strength						1,4				1,4		
	Unconfined strength						1,4				1,4		
	Weight density												
Spread footings	Bearing												
	Sliding												
Driven piles	Base											1,7/1.5	1,3
	Shaft (compression)											1.5/1.3	1,3
	Total/combined											1.7/1.5	1,3
	Shaft in tension											2.0/1.7	1.6
Bored piles	Base											2.0/1.7	1,6
	Shaft (compression)											1.6/1.4	1,3
	Total/combined											2.0/1.7	1.5
	Shaft in tension											2.0/1.7	1.6
CFA piles	Base											As	1.45
	Shaft (compression)											for	1.3
	Total/combined											bored	1.4
	Shaft in tension											piles	1.6
Anchors	Temporary											1,1	1,1
	Permanent											1,1	1,1
Retaining walls	Bearing capacity												
	Sliding resistance												
	Earth resistance												
Slopes	Earth resistance												

indicates partial factor = 1.0

Figure: 2.11: Factors adopted by the United Kingdom for Design Approach 1.

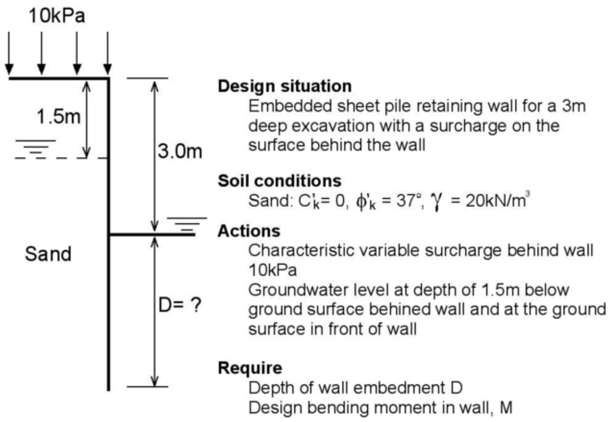


Figure 2.12: Embedded retaining wall considered in the Dublin workshop of 2005

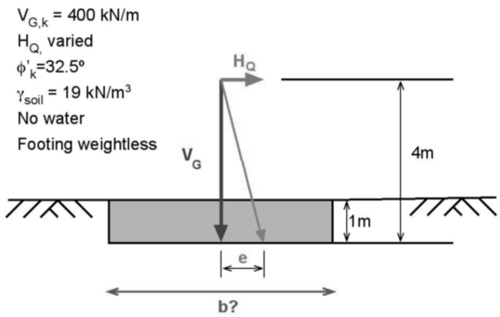


Figure 2.13: Inclined, eccentric loading on a footing

1.35 to the permanent vertical action if it is unfavourable (red dashed arrow), or 1.0 if it is favourable (red solid arrow). This is consistent with EN1990 and the Eurocodes for structural design, and in principle it is also required for DA2 and DA3.

Figure 2.15 shows similar diagrams for Combination 2 of DA1 and for DA2 and DA2*. In DA2*, for consideration of bearing capacity, the horizontal and vertical loads are combined into a single resultant before the load factors are applied, so the inclination and eccentricity of the design action effect takes no account of the separate factors on permanent and variable load, or of the possibility that the permanent vertical load is favourable.

The results of calculations of the required footing width for varying ratio H_k/V_k are shown in Figure 2.16, using the bearing capacity equations in Annex D of EC7. Results for DA1 and DA2 are fairly similar, and DA3 is more conservative, as expected. However, DA2* gives a markedly less conservative, and so less safe design. In Figure 2.17, the factors of safety available on the single quantities (a) γ_φ on $\tan\varphi'$ and (b) γ_Q on H_Q have been calculated using the footing widths obtained by DA2*. This shows that DA2* is equivalent to a single factor of safety slightly greater than 1.2 on $\tan\varphi'$ or a variable factor on H_Q which falls to about 1.17 at $H_{Q,k}/V_{G,k}$ of 0.4, the approximate limit of this ratio for sliding (EC7 allows $\delta=\varphi_{cv}'$ for concrete case on the ground). As the only safety factor in the calculation, a value of 1.17 on the variable load is remarkably low; a factor of 1.5 is generally applied to variable loads..

The proponents of DA2* argue that the results it obtains are in line with previous experience. However, the author submits that: (a) it is unlikely that there is a significant database of footings which have, in practice, been loaded to the high H/V ratios considered here (though it is very important that footings which have to be designed for high ratios are reliable); (b) the approach taken for bearing capacity is inconsistent with that taken for sliding, which is likely to cause confusion; (c) the approach taken for bearing capacity is inconsistent with that taken for structural design, which is also confusing; (d) as noted above, as the only safety factor in the calculation, a value of 1.17 on the variable load is remarkably low.

In practical use, design rules are often pushed to extremes not anticipated by code drafters. For this problem, it is desirable to check what happens if the lever arm of the horizontal load is larger. It is assumed here that structural designers would factor the vertical and horizontal actions independently. Figure 2.18, for a force at 10m above the base, shows that at a relatively modest ratio H_k/V_k= 0.13 the eccentricity of the resultant load used in structural design exceeds the half width of the footing derived by DA2*, implying that the resultant load passes outside the base of the footing, which cannot give equilibrium. This will clearly cause consternation and confusion to the structural designer, and illustrates the inconsistency of DA2*.

	DA 1
	Comb 1
V_G	1.35 or 1.0
G_Q	1.5
tan φ'	1.0
R (bearing)	
R (sliding)	

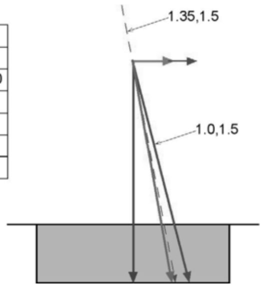


Figure 2.14: ULS design resultant actions derived for DA1.

	DA 1		DA 2	DA 2*	DA 2*
	Comb 1	Comb 2		bearing	sliding
V_G	1.35 or 1.0	1.0	1.35 or 1.0	1.35	1.0
G_Q	1.5	1.3	1.5	1.35 (1.5?)	1.5
tanφ'	1.0	1.25			
R (bearing)			1.4	1.4	
R (sliding)			1.1		

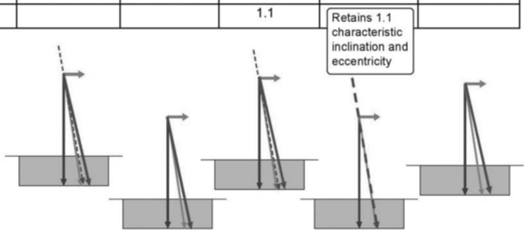


Figure 2.15: ULS design resultant actions derived for DA1, DA2 and DA2*

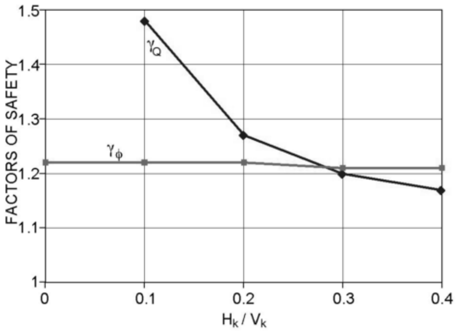
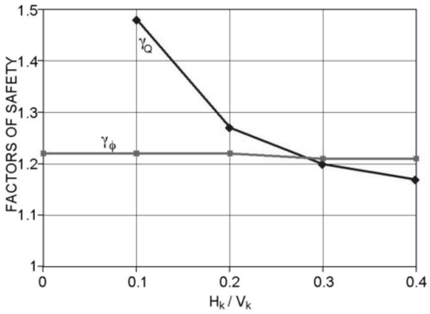


Figure 2.16: Footing widths calculated for ULS design

In contrast, Figure 2.19 shows that for DA1 the resultant load always lies within the width of the footing; Combination 2 is more critical than Combination 1, and so determines the width. Figure 2.20 shows that DA1 even accommodates a much more extreme lever arm for the horizontal load of 100m. This is guaranteed since the geotechnical design is checked for the same loading as the structure, consistently, in Combination 1, and for this extreme case it is Combination 1 which determines the footing width. For such an extreme case, much of the width of the footing might be redundant and the structure could be replaced by an A frame, but all the considerations noted above would still apply.



2.17: Total factors of safety implied by DA2 on tan□′ and HQ, taken singly.

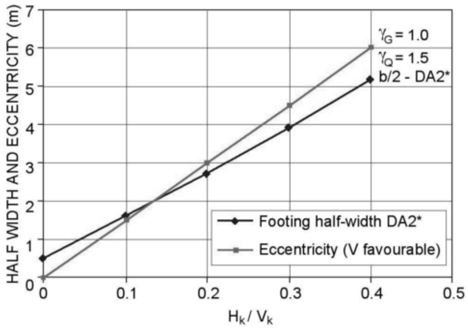


Figure 2.18: Eccentricities in DA2 and structural design, for 10m lever arm

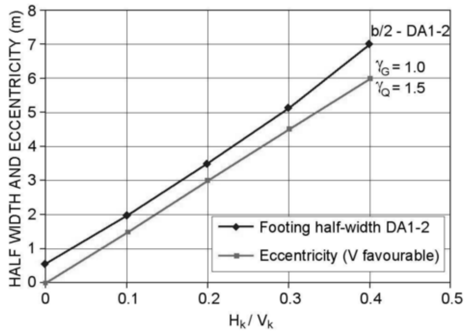


Figure 2.19: Eccentricities in DA1 and structural design, for 10m lever arm

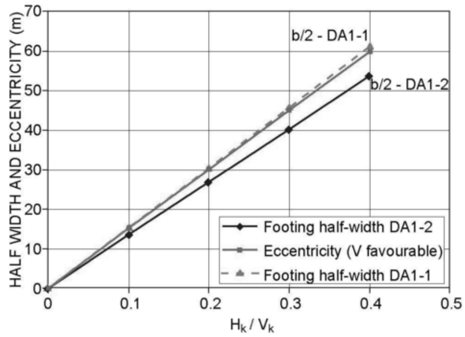


Figure 2.20: Eccentricities in DA1 and structural design, for 100m lever arm

2.7.2 *Piled foundations*

Traditional pile design has generally used overall factors of safety in the range 2 to 3, or even up to 4 in the case of tension loading. Often the lower end of this range is used when the design is verified by load testing; and often larger values are applied to base resistance than to shaft resistance. A major reason for the use of relatively high factors has been that they ensure serviceability as well as safety against ultimate failure. It is therefore necessary for developers of modern codes to decide whether they will perpetuate this situation or try to separate ULS from SLS. If a separation is attempted, it will not be possible, in many cases, to calibrate factors needed for ULS against existing practice, and SLS will often dominate the design, at least for larger diameter piles.

EC7 requires a "characteristic" pile resistance which is a "cautious estimate". In practice this can be derived from calculation based on ground testing or from load testing of piles, or a combination of the two. Indeed, EC7 specifically requires that calculation methods must be verified by testing, and test results must be checked by calculation {7.4.1(1)}, but it does not clarify how information from these two sources can be combined when both are used together. It also states that "For piles bearing in medium-to-dense soils and for tension piles, the safety requirements for the ultimate limit state design are normally sufficient to prevent a serviceability limit state in the supported structure" {7.6.4.1(2)}. Thus for piles, in contrast to other design elements, the code implicitly requires that factors for ULS design be chosen so that SLS will be satisfied. Hence the overall factors of safety adopted should probably be similar to previous practice.

Several European countries have reported that the partial factors proposed by EC7 imply overall factors of safety much lower than they would traditionally have used. The code is specific about the process of deriving design resistances from load tests, applying "correlation factors" ξ to the mean and lowest test results. Much less detail is given about calculation of resistances from ground strength tests, however. This means that each country has some freedom to specify in rather more detail how pile resistances will be calculated, and the values of partial factors may also be varied nationally, as for other design elements.

Bond and Simpson (2009) describe the background to the derivation of the factor values required by the UK National Annex to EC7 (BSI 2007). The partial factors γ have been increased compared with the values in the base version of EN1997-1 (see Figure 2.9); the correlation factors ξ have also been increased and additional "model factors" have been imposed in the derivation of characteristic shaft and base resistances from characteristic ground strengths. Both the model factors and partial factors γ have been varied according to the extent of load testing which supports the calculations. The overall effect for ULS calculation is shown to be consistent with previous design, and the characteristic values of resistance are suitable for serviceability calculations as, for example, in settlement reducing piles which might be loaded to their ultimate capacity in the working state. Example calculations are provided by Bond and Simpson (2009).

2.7.3 *Water pressures*

Application of factors to water pressure is fraught with difficulty and can easily lead to unreasonable situations. Orr (2005) reported calculations for the situation of potential hydraulic heave shown in Figure 2.21. He found that the calculated allowable height of water H could vary from 2.78m to 6.84m due to application of the same factors, taken from EC7, at different points in the calculation.

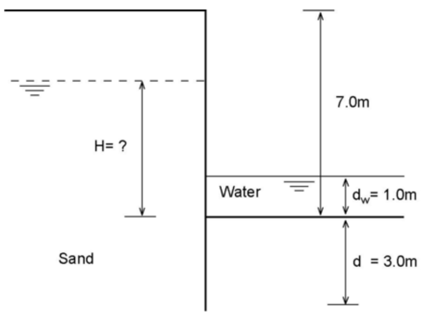
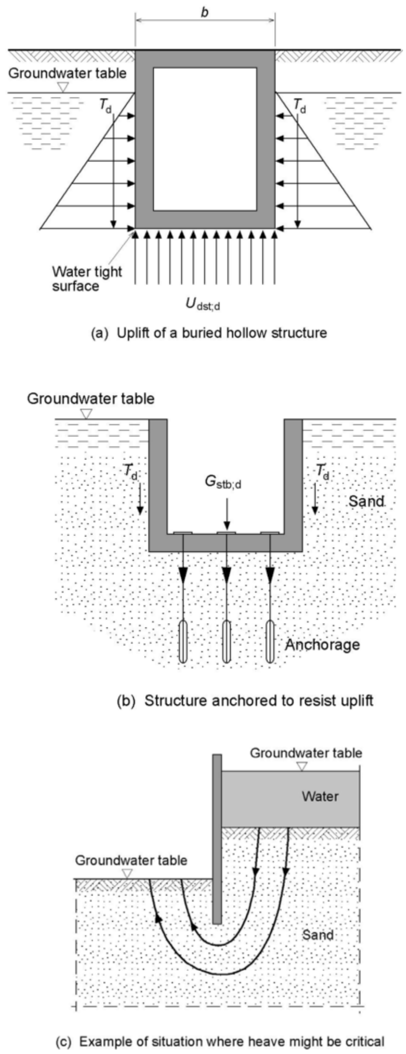


Figure 2.21: Hydraulic problem considered in the Dublin workshop of 2005

In design of structures in the ground, water pressure may constitute an additional load to be considered along with others, to be accommodated by strength requirements in structure and/or ground. In addition to this, two particular situations can be identified in which water pressures are principally balanced by other loads (weight of structures or ground): uplift failure and hydraulic failure, termed UPL and HYD in EC7, as illustrated in Figure 2.22. EC7 provides factors of safety to be used in checking these limit states, but it is not clear about where in the calculation they should be applied, leading to the confusion noted by Orr. In practice, these states are often complicated by the involvement of some element of structural or ground strength, such as the use of piles or anchors used to hold down the slab as shown in Figure 2.22(b).



(a)

(b)

(c)

Figure 2.22: (a, b) 'UPL' and (c) 'HYD' situations in EC7.

Modern codes tend to provide a single factor for "dead loads" or "permanent actions". It is arguable that static water pressure in the ground is a permanent action. In some cases, design can proceed, apparently reasonably, with a factor (eg 1.35) applied to water pressure, but in other cases applying a factor to water pressure is unreasonable. For example, Figure 2.23 shows a deep basement or shaft in ground with a high water table; increasing water pressure in the ground near the base of the shaft by a significant factor (eg 1.35) would represent a physically impossible situation and would constitute an unreasonably onerous design case.

The AASHTO code {10.6.3.1.1} requires that "bearing resistance shall be determined based on the highest anticipated position of groundwater level at the footing location", but it does not apply factors to water pressures for foundation or retaining wall design, despite factoring effective earth pressures. This appears to imply that in a situation where ground water pressure is dominant the design would rely almost entirely on the resistance factors in both the ground and the structure.

Eurocode 7 requires the designer to consider two situations. For ULS, the design should use "the most unfavourable values that could occur during the design lifetime of the structure", whereas for SLS "the most unfavourable values which could occur in normal circumstances". It also allows that design water pressures for ULS could be derived by applying a partial factor to characteristic water pressure or by raising the water level. Figure 2.24 compares water pressures for a 10m high wall with a 2m difference between these two levels. Starting from the "characteristic" water pressures (line A), it can be seen that merely applying a factor (obtaining line B) may underestimate the effect of raising a water level (line C); this is particularly the case when thrusts or bending moments are considered. On the other hand, factoring water pressures which are already "the most unfavourable values that could occur during the design lifetime of the structure" (obtaining lines D or E) seems unreasonable from both probabilistic and physical points of view.

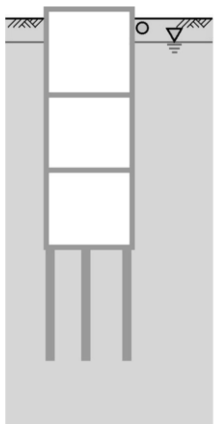


Figure 2.23: Deep basement or shaft in ground with a high water table

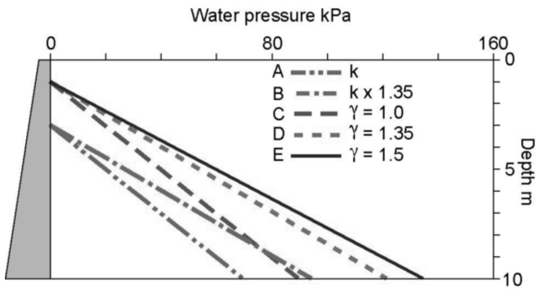


Figure 2.24: Water pressures for a 10m high wall

The UK National Annex to EC7 (BSI 2007) points designers away from simple factoring of water pressures, though it leaves the option open. It says that standard load factors "might not be appropriate for self-weight of water, ground-water pressure and other actions dependent on the level of water", and "the design value of such actions may be directly assessed in accordance with" the EC7 principles noted above, or alternatively "a safety margin may be applied to the characteristic water level".

In contrast, Bond and Harris (2008) provide two arguments in favour of factoring water pressures when designing retaining structures: (a) structural engineers have traditionally applied partial factors to retained liquid loads and ground water pressures, and (b) it is common for geotechnical engineers to perform calculations using unfactored parameters, including water pressures, and then to apply a factor of safety to the resultant structural effects, giving the same result as factoring both the effective earth pressures and water pressures.

Provisions for safety in relation to water pressure, both free water and ground water, remain under debate within the EC7 Maintenance Group. Given such uncertainty, making more than one check, effectively a parametric study, may be advisable. For EC7 Design Approach 1, the author has found that for most situations the following strategy gives reasonable results, though critical review is always necessary:

1. Apply DA1-1 using reasonably cautious water levels (most unfavourable in normal circumstances) and apply 1.35 to structural load effects - bending moments etc. Alternatively, in some cases, it is necessary to apply this factor to all permanent actions, including water pressures. Particular caution is needed if non-linear behaviour of structures is expected.
2. Apply DA1-2 using worst credible water levels (most unfavourable in design lifetime), unfactored.

2.7.4 *EQU*

Situations in which independent loads tend to cancel each other's effects were discussed above when considering working state design. In a partial factor approach, the design is generally made robust by applying different factors to the two loads. A further problem is sometimes encountered when two loads which are not independent, but arguably two parts of the same load, have a cancelling effect.

Eurocode "Basis of design", EN1990, considers that such loads come from a "single source", and notes: "For example, all actions originating from the self weight of the structure may be considered as coming from one source; this also applies if different materials are involved." When there is good reason to believe that the two loads must occur together, it seems unreasonably severe to factor them separately. Nevertheless, some allowance for variability of loads also appears necessary. EN1990 terms this situation the "EQU limit state" and requires a check by applying small load factors, generally 1.1 and 0.9, to the two parts of the single source load.

Schuppener et al (2009) have debated the significance of EQU to geotechnical design, presenting several alternative approaches without an agreed conclusion. In Figure 2.4(a), the approach to this issue determines the design bending moment in the column and changes the design loading in the piles, possibly putting one of them into tension; in Figure 2.4(b) it affects the design load in the anchor. The author's opinion is that "the whole system, both structure and geotechnics, should be designed for all the relevant design situations or load cases"; alternative approaches are inevitably more confusing and run the risk of leaving gaps between design cases, in which either a critical situation is not accommodated or design appears impossible.

2.8 *Economy and sustainability*

Codes influence both the safety of constructions, probably their primary aim, and their economy of constructions. With modern appreciation of the need to reduce usage of materials and energy, it is clearly desirable to produce more economic designs, which might be assisted by reductions in values of partial factors, where possible. It is commonly stated that geotechnical failures, ultimate or serviceability, are rarely caused by having factors of safety slightly too low. It is also reported in Eastern European countries that the factors proposed in EC7 (as published by CEN) lead to designs which are uneconomic compared to their previous practice, which they found satisfactory. This suggests that the proposed factors might be reduced, and so a proposal has been put to CEN to investigate further the designs used in Eastern Europe, with the aim of reducing the proposed factors. If it is possible, it will be necessary to compare records for safety and durability between societies adopting different designs. Hopefully, common international codes will provide a framework in which this might be achieved.

Individual factors must be reduced with care since factors on one variable contribute to the overall robustness of the design, perhaps giving unanticipated benefits . For example, the public enquiry into the Nicoll Highway collapse in Singapore (Magnus et al, 2005) noted that one of the contributory causes was factors of safety lower than code requirements. While this was not the prime cause of the failure, Simpson et al (2008) have argued that use of standard factors would probably have prevented collapse, mitigating the effects of more serious deficiencies. Further more, while reduced factors might not lead to collapse, they could result in more durability problems in later years. It is therefore very important that large scale surveys of performance are undertaken as an aid to introducing reductions in code factors.

Both economy and safety should be aided by more realistic evaluation of the necessary resistance of structures and ground. To this end, the Japanese development of Performance Based Design aims to provide a framework which incorporates limit state design and is particularly applicable to seismic situations.

Probably the biggest influence geotechnical codes can have on both safety and economy, however, is to emphasise the importance of good geotechnical process, desk study, ground investigation and choice of construction type. Calculations and factors, necessary for analysis and design, are important, but secondary. To quote EC7: "It should be considered that knowledge of the ground conditions depends on the extent and quality of the geotechnical investigations. Such knowledge and the control of workmanship are usually more significant to fulfilling the fundamental requirements than is precision in the calculation models and partial factors."

3 DEEP FOUNDATIONS

3.1 *Introduction*

The 2005 ICSMGE Osaka conference included a State-of-the-Art paper by Mandolini et al (2005) titled "Pile foundations: experimental investigation, analysis and design" and a further paper by Randolph et al (2005) titled "Challenges of Offshore Geotechnical Engineering". In Mandolini et al (2005), the authors reviewed both single pile behaviour and pile group behaviour, considering both experimental results and analysis. The paper by Mandolini and his co-authors remains a thorough review of these aspects of pile design and analysis. In Randolph et al (2005) a summary of offshore ground investigation is presented along with design and construction aspects of piled foundations and spread footings (suction caissons, gravity structures etc). Four years further on, the ISSMGE committee for the Alexandria conference has asked for this short contribution on deep foundations as part of a more general paper on design and analysis; the committee suggested that large diameter or "mega" piles could be discussed. This contribution provides an update on design processes for single piles in soil and rocks using design processes from both offshore and onshore developments with bias towards large diameter piles and vertical loading.

In addition to the sections on load-carrying capacity, comments on sustainable aspects of pile design are provided in view of the recent progress in this area and the lack of inclusion in previous ICSMFE state-of-the-art reports.

Prior to presenting a limited number of pile design methods it is worthwhile setting the scene for acceptable design. Eurocode 7: Geotechnical design - Part 1: General rules (BSI, 2004) provides reasonable requirements for acceptable design:

The design [of piles] shall be based on one of the following approaches:

- [Method 1] the results of static load tests, which have been demonstrated, by means of calculations or otherwise, to be consistent with other relevant experience;
- [Method 2] empirical or analytical calculation methods whose validity has been demonstrated by static load tests in comparable situations.

Within this report attention is paid to the second of these two methods where pile load test data is used to develop design methods. While the first method (design by site specific pile test data) is not presented herein, it clearly remains a valid approach provided measured pile resistances can be justified by calculation methods derived in keeping with Method 2.

3.2 *"Mega" piles: an introduction to vertical design*

Where appropriate, large diameter piles are used in onshore construction for infrastructure projects or high-rise building development both of which generate high loads. The upper limit of such piles is typically 1.5m in diameter, occasionally 2.1m or 2.4m in diameter, due to limitations on the size of construction plant. In offshore locations "mega" piles with diameters in excess of 5m are now possible owing to larger construction plant and the economics of construction (speed of construction is critical to economy off-shore). The design of such large diameter piles is presented below with a bias towards developments in offshore design practice.

Traditional methods of pile design rely heavily on observation leading to empirical equations for assessment of axial (geotechnical) capacity. This is still true, even though, as pointed out by Randolph (2003a), the scientific understanding of soil behaviour in general and around piles is continually improving. Such improvements in understanding allow observed pile behaviour to be better explained, thereby allowing for better design. Design based on empiricism is nevertheless the norm for assessment of pile capacity under axial loading, although theoretically based equations are sometime used for assessment of base resistance in some instances.

This general summary of the current state of design is illustrated by the much used API (2000) "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms" document where much of design guidance relies on empirically assessed resistance similar to Equations 3.2 and 3.3 below. (It is understood that a revised version of the API document will be released in due course which will be updated to included much of the work referenced below for driven piles offshore.) The following review of pile design shows that while empiricism remains at the forefront of pile design, soil mechanics theory is continually developing and providing important insights into the assessment of pile capacity and behaviour. Soil mechanics should be seen as a route to improving our understanding of pile behaviour and, with recourse to best practice design methods, to informing ground investigation techniques.

In the following, care is taken to present equations and data for ultimate conditions. Where "allowable" or working capacities are quoted, these are clearly identified. When choosing the base soil parameters (e.g. characteristic values in the Eurocode design) the designer must accommodate the basis of the code adopted. For example, an average value of shear strength may be appropriate to one code while in another the parameter may need to be a cautious estimate. Failure to understand the basis of the code will result in inappropriate and potentially unsafe designs.

In assessment of the ultimate limit state (ULS) many conventional design processes protected the serviceable limit state (SLS) for conventional structures by limiting allowable working load to a percentage of the skin friction (e.g. skin friction / 1.2, in addition to a factor of safety on total capacity) or by ensuring mobilisation of base resistance is small. As codes become more sophisticated and as construction extends beyond historic limitations (e.g. large diameter piles), there is a greater need to assess pile settlement as part of the design process with attention paid independently to the rate of mobilisation of shaft and base resistances. Clearly where there is direct comparable experience (loading, foundation size and ground conditions) the requirement for explicit assessment of settlement is reduced. Where such comparable experience is absent then explicit assessment of settlement should be the norm.

As a final introductory comment, no discussion on the structural capacity or durability of piles is provided herein. Such considerations are clearly required for a design in practice and should accommodate applied actions and construction tolerances as well as changes to the structure or dimension of the pile with time (e.g. corrosion of steel piles).

3.2.1 *Pile design based on offshore experience: soil*

The following paragraphs present a summary of offshore pile design for silica sands and clays. Comment is made on pile capacity in carbonate soils in a following section.

Conventional design of piles usually adopts an equation as follows:

$$Q = (Q_b + Q_s) = (A_b q_b + \bullet A_s q_s) \tag{3.1}$$

Where Q is pile resistance and q is limiting or failure stress, A is area and subscripts b and s are base and shaft respectively. The method for calculation of q_b and q_s has, in the past, been relatively simple and commonly took the form of:

In sand:

$$Q = A_b q_b + \bullet A_s q_s = \tfrac{1}{4}\ D^2 N_q\ '_v + \ D \bullet (K\ '_v \tan\ )dL \tag{3.2}$$

and in clay:

$$Q = A_b q_b + A_s q_s = \tfrac{1}{4}\ D^2 N_c\, s_{u\,base} + \ D \bullet (\ s_{u\,L})dL \tag{3.3}$$

where the terms are as follows:

K earth pressure coefficient on the pile shaft
  angle of interface friction (often expressed as a fraction of the shear strength, e.g. 2⁄3 φ)
  shaft adhesion factor applied to s_u
D pile diameter
L length for assessment of pile shaft friction
s_u undrained shear strength of the soil (clay). Subscripts “base” and “L” illustrate the likely different values in s_u that will be used in the calculation at the base and incrementally along the shaft.

These equations are populated with a combination of theoretical (Nq, Nc) and empirical (K, α) parameters. When using such equations there is significant guidance on the choice of the design constants and limiting stresses for situations within bounds of comparable experience. Such guidance has been distilled into books such as Poulos and Davis (1980) and Fleming et al (2008) among others. Whilst these equations and their empirical parameters have come about from experience and remain of practical importance, they do not provide adequate confidence for situations where there is no comparable experience. Such situations include new construction techniques, sites with no previous pile performance data, changes in pile dimensions compared to test data and where pile construction is in new strata. Taking these conditions in turn:

- Where new construction techniques are adopted there is little option but to carry out high quality instrumented load tests to investigate pile behaviour. The results of such tests should be interpreted so as to distinguish between pile shaft and base behaviour (load settlement response). The results should also be assessed against the best available calculations in order to have confidence of the validity of test data. Added confidence is gained as more piles are tested, a process formalised by Eurocode 7 Part 1 (BSI, 2004).
- Where a new location is being developed, an understanding of the soil is critical with, as a minimum, assessment of geomorphological and geological situations (ground model), mineralogy and soil consistency (relative density, strength and stiffness data) being required to correlate to other locations.
- Where pile size is larger than test pile diameters, then interpretation of test data must allow for the effect of diameter on base and shaft capacities and overall settlement to derive ULS and SLS conditions. As noted in sections 3.2.2 and 3.2.5 below the measured pile shaft resistance (stress) varies as pile diameter increases thereby not allowing linear extrapolation from small to large diameter piles when assessing shaft capacity.
- Finally, where piles are constructed in new strata, the behaviour of the strata during pile construction and subsequent loading must be understood.

As an example of the above comment on soil strata, pile base capacity calculated using a constant value of φ' and a recognised bearing capacity factor (N_q) such as Berezantzev et al (1961) will fail to recognise the subtleties of soil behaviour at varying stress levels as illustrated by Bolton (1987), Fleming (1992) and as extended by Cheng (2004). In the context of pile base bearing capacity, higher levels of confining stress result in a reduction of the ability of the soil to dilate. This prevents the peak angles of friction seen on samples tested at low effective stress being mobilised. This observation is valid for siliceous sands but becomes critical when softer soil grains are involved due to early onset of particle crushing and further suppression of dilatancy (Klotz and Coop 2001).

3.2.2 *Offshore: Driven piles – a CPT approach*

The environment in which offshore piles are constructed combined with the requirement to carry large loads has resulted in offshore piles being significantly larger than their onshore counterparts. Driven tubular piles installed offshore can be 5m and larger in diameter, dwarfing typical construction onshore. The viability of such large piles is linked to progress in pile installation technology and to the research into axial pile behaviour resulting in new design methodologies as referenced below. Whilst these new design methods are driven by the requirements of offshore construction, they can equally well be used to inform onshore pile design when similar construction techniques are used and ground investigations carried out. They may also be used as starting points for development of methods appropriate to onshore practice. The design methods typically use (sea-bed driven) CPTs to provide a profile of soil resistance together with laboratory testing to provide further parameters (density, interface strengths, magnitude and variation of Ko etc.). Where CPT data is not continuous appropriate averaging techniques must be used to generate a continuous profile.

The UWA-05 (University of Western Australia 05, Lehane et al., 2005) method is used to illustrate the developments in pile design in siliceous sand, albeit, as noted by Monzon (2006), alternatives exist in the form of ICP-05 (Jardine et al) and NGI-05 (Clausen et al, 2005). The UWA report deals with piles installed (driven) in silica sands only. All methods mentioned above differentiate between closed and open ended driven piles owing to the different strain fields which develop during driving.

<u>Base resistance</u> for close ended driven piles in sand is related to CPT cone resistance q_c as follows:

$$Q_b = A_b\, q_{b0.1c} \tag{3.4}$$

$q_{b0.1c}$ = $0.6q_{c\ ave}$ (for jacked piles the factor 0.6 can be increased to 0.9)
$q_{c\ ave}$ the average value of q_c over a distance 1.5 times the pile diameter above and below the pile tip (further reference is provided in Xu and Lehane, 2005).

The subscript 0.1c denotes the mobilised resistance at a settlement of 10% (0.1) of the base diameter for a closed ended (c) driven pile; the equation is clearly empirical.

For open ended piles experience (Lehane and Randolph, 2002) shows that in siliceous sand, piles loaded under static conditions tend to plug when the height of plug inside the pile at the end of driving is greater than 5 times the pile internal diameter ($5D_i$). This observation was made on piles of up to 1.5m diameter, care must be exercised for piles with greater diameters and additional research is required to allow general adoption of this observation. The result of this is that shaft friction is generally linked to the external face of the pile only and base capacity is modified as follows:

$$Q_b = A_b q_{b0.1o} \qquad (3.5)$$

where

$$\frac{q_{b0.1o}}{q_{cave}} = \left[0.15 + 0.45\left(\frac{D*}{D}\right)^2\right] \qquad (3.6)$$

The subscript "0.1o" denotes the mobilised resistance at a settlement of 10% (0.1) of the base diameter for an open ended (o) driven pile. While it is assumed that the pile plugs, the open ended pile "capacity" is less than that of a closed ended pile due to the higher stiffness exhibited by the closed ended piles compared to an open ended pile. This observation in stiffness is a common thread seen in other empirical based design methods as in Section 3.2.3 below. It is also consistent with previous comments in Section 2.6.2 where partial factors for ULS pile design generally provide a degree of comfort for SLS considerations. D* is the effective diameter of the pile's base and is defined as (all pile diameter dimensions in metres):

$$\frac{D*}{D} = \left(1 - \min\left\{1, \left[\frac{D_i}{1.5}\right]^{0.2}\right\}\left(\frac{D_i^2}{D^2}\right)\right)^{0.5} \qquad (3.7)$$

For open ended piles $q_{c\ ave}$ is the average value of q_c over a dimension of 1.5 times D* above and below the pile tip and not the external diameter, D, as for close ended piles. In Equation 3.7 the "min" value is referred to as the "final filling ratio" FFR) and is, in the absence of direct measurement of the plug length inside the pile, an approximation allowing for the plug length. It should be noted that this FFR approximation only holds good for situations where the pile tip has penetrated to at least 5 times the internal diameter of the pile into the sand layer. Where this is not the case, assessment of non-plugged pile capacity will be necessary.

Value of D^*/D are presented in Figure 3.1 for D_i /D in the range of 0.98 down to 0.0. D_i /D = 0 for a closed ended pile and 0.98 for a thin wall pile. The limiting conditions occur when the internal diameter equals 1.5m as can be seen in Equation 3.7 above. Figure 3.2 shows the variation in the ratio of end bearing stress (average over the full cross sectional area) and cone resistance in the vicinity of the pile tip with pile diameter for a range of Di/D values; for closed ended piles the ratio is 0.6, falling to less than 0.18 for relatively thin walled tubes.

As would be expected, for a zero dimension of the internal diameter D_i (i.e. a closed ended pile), the value of Q_b is the same as a closed ended value in Equation 3.4 above.

Assessment of pile shaft capacity for open and closed ended piles has been combined into a single method in siliceous sands (Lehane et al, 2005b). This is possible due to the tendency of open ended piles to plug during static loading thus negating the need to assess skin friction on the inside of the piles tube.. While the method of assessment of pile shaft capacity is the same for open end closed ended piles, closed ended piles mobilise high resistances due to higher radial stresses acting on the pile shaft resulting from the displacement nature of these

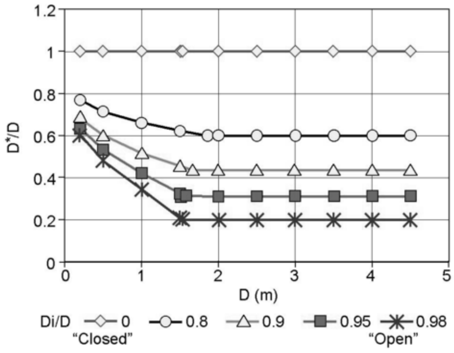


Figure 3.1 Relationship between Effective Diameter, pile internal diameter and pile external diameter.

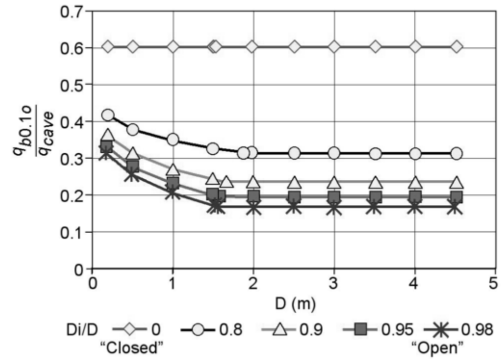


Figure 3.2 Relationship between end bearing stress on full pile diameter and average cone resistance at pile tip with pile diameter as a function of Di/D.

piles. Measured pile shaft friction is linked with cone resistance and the fatigue induced at the interface between pile and sand by pile driving as well as the interface friction (sand to pile) and radial effective stress acting on the piles shaft due to the displacement nature of the pile. The empirical correlation between pile shaft friction q_s and cone resistance q_c is expressed in terms apparently dissimilar to the shaft component in Equation 3.2 above. The term $K\sigma'_v$ (= σ'_h) in Equation 3.2 is replaced by a function of q_c which accounts for horizontal effective stress working on the pile shaft. The horizontal effective stress is a result of the post-installation conditions at the pile face combined with the dilation of the soil adjacent to the pile which is mobilised during pile loading. The latter term can be compared to the enhanced resistance derived in a constant normal stiffness shear box test, where dilation results in an increase in confining stress and therefore shear resistance. For large diameter piles the component of shaft friction derived from dilatancy of the soil at the pile interface is relative small allowing a simpler expression for shaft friction to be presented (Lehane et al, 2005b):

$$q_s = a.q_c A_{rs}^{\ 0.3} \times \left[\max\left(2, \frac{h}{D}\right)\right]^{-0.5} \times \tan\delta_{cv} \qquad (3.8)$$

q_s = $'_h$ x Fatigue factor x Coefficient of friction
a a constant equal to 0.03 for compression piles and 0.0225 for tension piles.
A_{rs} the effective area ratio for the shaft.
h distance of the section of pile shaft above the pile tip (factor allows for fatigue during installation. For long piles the top section of the pile has large value of "h" resulting in reduced skin fiction at that location compared to locations further down the piles (assuming similar values of q_c).

$_{cv}$ constant volume angle of interface friction (sand on steel), correlated to the mean particle size (approx = 23° for D_{50} = 1mm rising to an upper bound of 28.8° for D_{50} = 0.2mm).

A_{rs} is equal to 1.0 for closed ended piles and can be calculated for open ended piles from the following: $A_{rs} = 1 - IFR_{mean} (D_i / D)^2$ where IFR_{mean} is the incremental filling ratio and, in the absence of measured data is equal to $IFR_{mean} = \min(1,(D_i /1.5)^{0.2})$. Values of $A_{rs}^{0.3}$ are presented in Figure 3.3 where it can be seen that a thin walled pile (D_i/ D = 0.98) has approximately 40% of the shaft friction of a closed ended piles (D_i / D = 0)

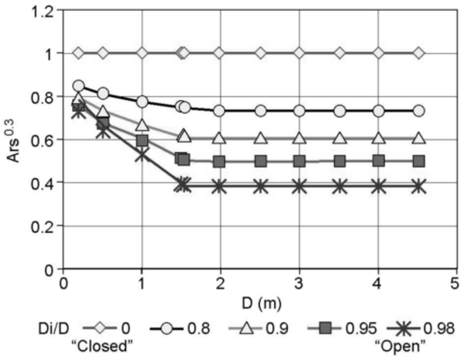


Figure 3.3 relationship between Effective Area Ratio (^0.3) and diameter for various Di/D values.

It is noted that Equation 3.8 is appropriate for use with driven piles. Where piles are jacked then a similar approach can be taken except that the fatigue factor ($\max(2,h/D)^{-0.5}$) must be modified. The approach presented by Jardine et al (2005) provides more data.

In order to allow for dilatancy the reader is referred to the original paper to allow economic design of small diameter piles. As noted above, the dilatancy effect does not increase the shaft resistance significantly (more than 10%) for all but small diameter piles (less than about 0.6m external diameter).

The above method of pile design is based on the results of load tests on a large number of relatively small diameter piles, as is fully acknowledged by the authors of the pile design method. The potential benefit of carrying out maintained load prototype scale pile tests is worth investigation as suggested by White and Bolton (2005), and should be pursued where possible (e.g. for large off-shore wind farms where the benefits of scale may justify the initial cost of testing).

While the above suggests that design development has progressed significantly in recent years it is also noted that significant research continues. This is well illustrated by a recent research project led by the Norwegian Geotechnical Institute on pile testing and design. The project aims to test relatively small diameter (0.3 to 0.4m diameter) driven open tube piles in up to 6 varying soil types (see comments on the need to test large diameter piles above). At each site, piles will be installed and load tested over a period of 2 years to identify how pile capacity increases with time from construction to two years post construction. Investigation of how loading affects pile capacity will also be carried out. While increase of pile capacity with time is acknowledged, the research aims to provide greater clarity on the issue. Since pile testing is usually carried out shortly after construction, design methods typically predict pile capacity at 10 to 20 days after construction.

3.2.3 *Onshore: Bored piles - a pressuremeter approach*

Compared to offshore development, for a majority of onshore developments there will be increased constraints in terms of noise and vibration. The result of this is that, for urban development at least, large diameter piles tend to be bored and cast-in-place rather then driven. Whilst onshore and offshore pile sizes and loading conditions are different, the requirement for overall economy is comparable albeit the manner in which economy is achieved is not necessarily the same. For offshore construction the cost of plant (and thereby time on site) is a very important factor whilst for onshore conditions material usage has a larger impact on overall cost.

When considering the technical aspects, rather than the economic aspects of design, the soil around onshore piles experiences disturbance in a similar manner to that of offshore piles for similar pile types (bored, driven etc). Installation processes modify in-situ stresses, and possibly ground strengths due to remoulding and shearing, to such an extent that soil mechanics alone is again rarely able to predict pile performance accurately. Hence, in the geotechnical design of onshore piles constructed in soil, empirical procedures dominate. An illustrative example of this is well documented in French design practice, now enshrined in Eurocode 7 Part 2 (BSI, 2007a), using the Ménard Pressuremeter limit pressure to assess both limiting shaft friction and base resistance for all main pile types in a wide variety of soil types. Such design is useful as it is based on a large set of pile observations and is appropriately conservative. It is does not, however, lead to an understanding of pile behaviour.

Published design guidance (BSI 2007a) can be supplemented by site specific correlations to optimise pile design and to accommodate different methods of pile installation. An example of this is bored-base-grouted piles in dense sand (Thanet Sand) used in East London (UK) for tall buildings and occasionally in central London where deep infrastructure prevents founding at shallow depth (Yeow et al, 2005). Early design procedures were based on pile load tests (Troughton and Platis, 1989, Chapman et al, 1999). Pile base capacity was correlated with mean effective stress rather than the more usual vertical effective stress. Variations in the "apparent" relative density of the sand with depth (corrected SPT blow count reducing with depth) along with increasing fines content with depth could not, with confidence, be used to assess pile base capacity variation with depth. This resulted in the need for a more robust method of profiling pile end bearing resistance with depth.

The Menard Pressuremeter approach (Nicholson et al 2002) was adopted and equipment extended to allow testing to 10MPa to allow the engineering implications of the changes in the sand with depth to be accommodated in design (typical equipment is limited to 5MPa or less and would not allow direct limit pressure measurement). The accepted equation for pile base capacity using the Ménard Pressuremeter limit pressure is:

$$q_b = k_s (P_{lim} - {}_{h0}) + {}_{h0} \quad (3.9)$$

k_s empirical factor
P_{lim} limit pressure
$_{h0}$ horizontal total stress

The value of k_s is purely empirical as illustrated by the values of 1.6 for bored piles and 3.2 for driven piles in dense sands (BSI, 2007). This derives from differences in pile base stiffness for loading to 10% of pile base diameter (driven piles are stiffer then bored piles in the sandy soils). For bored-base-grouted piles based in Thanet Sand, a value of k_s of 2.5 was found to correlate with load test results. The value of 2.5 lies between 1.6 (conventionally bored) and 3.2 (driven) as suggested by Nicholson et al (2002). The value is clearly linked to the nature of base grouting and as with all empirical factors is very much method related and cannot be taken as universally applicable.

As with all empirically based design methods the parameter (limit pressure in this case) is only valid if the conditions pertaining at the time of ground investigation test are similar to those governing the design of the foundation. If the conditions (notably effective stress regime in the soil) vary between the time of the ground investigation testing and the design condition, these changes must be accommodated in the design. Such changes result from basement excavations or changes in groundwater table. Whilst the design approach is clearly empirical the method of correction must be based on something more substantive in order to predict pile capacity at the design condition. For the case of piles in sand, the method by Troughton and Platis (1989) corrected base resistance by the ratio of mean effective stresses ($_m$') at the design condition compared to the testing condition based on measured pile performance. Using this approach then the corrected pile base capacity is:

$$q_b \;=\; (\;'_{m\ design} \,/\; '_{m\ GI})\;(k_s\,(P_{lim\ GI} - \;_{h0\ GI}) + \;_{h0\ GI}) \qquad (3.10)$$

where subscripts "design" and "GI" indicate the working condition of the pile and the conditions existing at the time of the measurement of limit pressure.

Menard Pressuremeter limit pressure can similarly be used to predict shaft capacity as described in BSI (2007a) where empirically derived graphs are presented plotting limit pressure against shaft friction as a function of soil type and density. Where a reduction in effective stress occurs between pressuremeter testing and the design condition then it is possible, for conventionally bored piles in sands at least, to correct the limit pressure as a function of changes in horizontal effective stress ($\approx \delta p_{lim} = p_{lim\ test} \times \delta\sigma_h' / \sigma_h'$). Assessment of the initial σ_h' requires a knowledge of stress history (Mayne and Kulhawy, 1982) or by direct measurement. Change in σ_h' ($\delta\sigma_h'$) may be made assuming $\delta\sigma_h' = (\nu' / 1\text{-}\nu') \times \delta\sigma_v'$, where ν' is the drained Poisson's ratio (Troughton and Platis, 1989). A check on the final ratio of $\sigma_h' / \delta\sigma_v'$ is necessary such that $\sigma_h' \leq 1/ K_{o\ NC} \times \sigma_v'$ where $K_{o\ NC}$ is the normally consolidate value of K_o ($\approx 1-\sin\varphi'$) unless pile testing shows higher limiting values (such as $\sigma_h' \leq K_p \times \sigma_v'$). It would be reasonable to assume that such an approach is appropriate only to a reduction in limit pressure and not an increase. Such changes in effective stress could be due to wide excavations or changes in groundwater level following completion of the ground investigation.

While pile design is dominated by empiricism our understanding of soil behaviour is more firmly rooted in science and theory. This is ably demonstrated by Ventouras and Coop (2009) who investigated the case of reducing pile base capacity with depth by means of triaxial testing. The tests were carried out at stress levels appropriate to a loaded pile bases in order to investigate the how the variations in soil grading and particle mineralogy could effect pile capacity. The results showed that the increase in fines content with depth was the dominant factor resulting in reduced pile base capacity with depth due to reduction in the ability of the soil to dilate during shearing. While the conclusions of the research are not such that pile capacity can be predicted for conventional design, they do provide better understanding on how piles behave. Such understanding results in better and safer design.

3.2.4 *Piles – Carbonate soils and chalk:*

The above description of offshore driven piles and onshore bored piles is related to silica dominated sands. Where carbonate sands are present (often lightly cemented) experience shows that bored piles (drilled and grouted piles) may be preferable to driven piles. This is due to the likelihood of low confining stresses acting on the shafts of driven piles caused by the local collapse of the adjacent soil (installation damage) and the ability, thereafter, of the soil to arch around the pile. Such behaviour is well documented from the Rankin field of Western Australia (King and Lodge, 1988) where the cost of remedying failed driven piles came to A$300m at the time (at current prices about US$500m excluding costs associated with loss of revenue). The impact of such soil-structure interaction is that drilled and grouted piles (or possibly driven and grouted piles) are a more suitable foundation type for such soils. Design recommendations, such as those of Randolph et al (2005) suggest a peak shaft friction ($q_{s\ (peak)}$) as per Equation 3.11:

$$q_{s\ (peak)} \bullet q_c\ (0.02 + 0.2\ e^{-0.04\ qc/pa}) \qquad 3.11$$

where p_a is atmospheric pressure (100kPa),

Similar behaviour is also well documented for piles driven into chalk (low and medium density chalk) where the limiting design shaft friction for driven piles is often 20 to 30% of the equivalent cast-in-place pile. Again this is due to the contractant nature of the remoulded chalk and the ability of the chalk mass to arch around the pile shaft (Lord et al, 2002).

For initial assessment of pile behaviour in chalk or carbonate soils, the reader is directed to Lord et al (2002) and Randolph (2005) respectively.

The above example highlights the need to obtain a thorough understanding of the effects of soil mineralogy and in-situ state when carrying out pile design. For designs being carried out at new locations and where the soil has not supported piles or indeed the type of pile being proposed, design must not rely only on empirical rules developed elsewhere for different conditions. An open mind must be maintained and the ability to load test piles must be incorporated into the programme and cost plan.

3.2.5 *Drilled and grouted piles – rock:*

Drilled and grouted piles are commonly used in near-shore locations where rock exists below the seabed and the construction of direct bearing foundations (gravity structures) is not cost effective (e.g. where there are significant soft sediments above rock head). These foundations are common for port structures, bridge foundations and wind turbines as well as for oil and gas industry related structures. Such foundations have become more common in recent years as a result of developments in piling technology allowing single piles to carry large loads (often limited by the structural capacity of the pile shaft). These positive developments have also coincided with heightened awareness of the dangers of construction utilising divers or working under pressure, as in the case of sunk caissons, further championing the case for drilled and grouted pile foundations. In parallel with advances in construction capability there have also been consistent advances in design, as presented below.

Shaft Friction: Classical design of rock-socket piles is empirically based (Rowe and Armitage, 1987, amongst others) and relates the unconfined compression strength of the intact rock to the ultimate shaft friction (q_s) as follows:

$$q_s \;=\; \;_{q}\;_{ci} \qquad (3.12)$$

or

$$q_s \;=\; \;_{c}\;_{ci} \qquad (3.13)$$

q_s unit shaft friction (MPa)

$_q$, $_c$ and are empirical constants

$_{ci}$ intact rock unconfined compression strength (UCS in MPa)

Values of $_c$ and are typically 0.2 to 0.4 and 0.5 respectively (Hovarth et al, 1980). The range in the value of $_c$ is related to the roughness of the rock socket.

The parameter α_q is shown in Figure 3.4 and is obtained by combining Equations 3.12 and 3.13, giving:

$$\alpha_q \;=\; \;_{c}\;_{ci}^{\;-1} \qquad (3.14)$$

Figure 3.4 shows the range of values α_q in this case for roughness in the range of R1 (smooth, indentations less then 1mm) to R3 (grooves 4-10mm deep, >5mm wide, spacing 50-200mm) as defined by Rowe and Armitage (1987). The plot demonstrates the non-linear relationship between shaft friction and rock strength. Further data is provided by Rowe and Armitage (1987) who suggested average values of shaft friction

constants (for relatively small diameter sockets, typically less than 0.6m diameter) as follows:

- $_c$ is equal to 0.4 for R1 to R3 sockets and 0.55 for R4 sockets (R1 to R3 as above, R4 socket quality is defined as grooves with depth and width greater than 10mm and spacing between 50 and 200mm).
- is equal to 0.57 for R1 to R3 sockets and 0.61 for R4 sockets

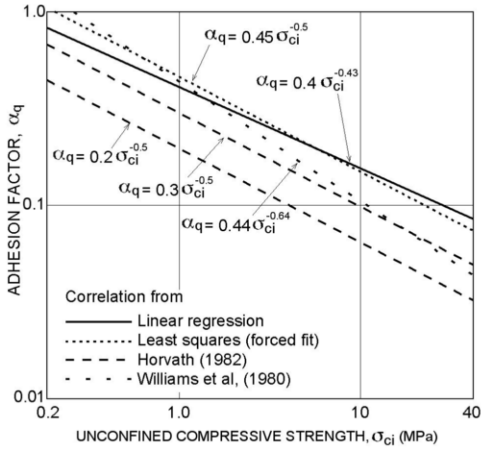


Figure 3.4 Relationship between shaft resistance and rock UCS for socket roughness in R1 to R3 range (after Rowe and Armitage, 1984). Note use of q_u for UCS of rock, σ_{ci} used elsewhere.

It is considered that such methods are useful for preliminary design but must be used with caution where no comparable experience is available (same rock, same pile size, same construction process). The values shown above are best fit values and have been derived from load test data. Use of such design constants within design codes such as Eurocode 7 Part 1 (EN1997-1) could be seen to fail the requirement that the design process returns a resistance that is "accurate or erring on the side of caution" (this requirement is not the same as a best estimate). The requirement results in the need to assess how a "characteristic" skin friction should be derived. Assessment of such characteristic values of skin friction may be obtained by adopting cautious estimates of both $_{ci}$ and $_c$. However, even when this is done several of the Eurocode 7 National Annexes (eg the UK national annex for Eurocode 7 Part 1: BSI, 2007b) require that a further model factor be applied to the values to acknowledge the uncertainty in the calculation and to arrive at pile resistances in keeping with previous national practice and expectation.

As noted above, the use of such empirical relationships is only acceptable when the design constants are available from comparable pile load tests, i.e. load tests on similar sized piles (socket size diameter and length) formed using similar equipment (drill bit detail and flush methodology) carried out in the same bearing stratum. Use of extrapolated data is inherently dangerous. Where, as is likely to be the case with most large diameter piles, good quality comparable pile load data does not exist then it is necessary to look to design methodologies which are theory based, and which have been checked elsewhere against pile load test data as required by Method 2 in section 3.1 above. Appropriate design methods should allow for variability in ground conditions, construction techniques and pile size by means of readily measurable parameters. These parameters are then used to populate design equations. Such an approach has been presented by Seidel and Collingwood (2001) where the observations made by previous researchers have found a more rigorous home. The method combines the effects of the following:

1. Rock mass stiffness;
2. Rock socket construction disturbance and cleanliness;
3. Socket geometry (length, diameter and roughness) (see Figure 3.5 below);
4. Rock strength (unconfined compression strength and the critical state angle of shearing resistance, ϕ'_{cv}, of a fissure as would be measured in a shear test)

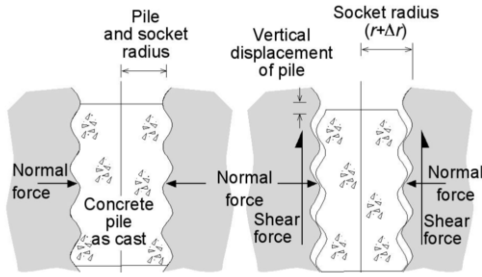


Figure 3.5 Illustrative model of pile rock socket displacement under axial compressive load (from Seidel and Collingwood, 2001, after Johnston and Lam, 1989)

These attributes will be considered in turn.

1. Beyond controlling the overall settlement in the rock mass, rock mass stiffness has a significant effect on ultimate skin friction by providing restraint to dilation on the pile/rock interface. Dilation occurs as a function of the socket roughness where shear displacement on the interface results in radial strains in the surrounding rock mass and corresponding increases in normal stress acting on the pile/rock interface. The constant normal stiffness value (K), is related to the rock mass modulus (E_m) and it is inversely proportional to socket diameter as shown in Equation 3.15:

$$K = 2E_m \;/\; [(1 + \acute{\nu}).D] \tag{3.15}$$

K	constant normal stiffness parameter
E_m	rock mass modulus
$\acute{\nu}$	Poisson's ratio
D	rock socket diameter.

It is the parameter K which controls the ability of the rock to resist dilation. Clearly, as seen in Equation 3.15, the larger the pile diameter the smaller the value of K. This is a similar effect as seen for driven piles in sand above where the beneficial effects of dilation are greatest for small diameter piles.

The rock mass modulus (E_m) can be measured directly (eg by dilatometer testing), or is more usually correlated with the rock UCS, Rock Mass Rating (RMR) or rock mass quality (US Army, 1994)

2. The construction process used to form the pile and the cleanliness of the pile shaft are important in assessing the efficiency by which the pile shaft can transfer load to the ground. High quality construction leading to a clean shaft provides highest resistance while low quality construction leaving smear results in low resistance. This variation in construction quality can be represented by the factor η_c as discussed below.
3. The rock socket geometry is critical in assessment of shaft capacity with rough sockets resulting in increased interaction between the pile and the surrounding rock (see earlier comments on socket resistance above for R1-R3 and R4 roughness from Rowe and Armitage, 1987). The socket roughness is expressed in terms of change in radius of the

socket wall (r). Seidel and Haberfield (1995a) present a theoretical design development of rock sockets incorporating a Joint Roughness Coefficient (JRC) and fractal mathematics which better describe the effects of scale (both of asperity height and geometry) of rock socket roughness. Such discussion is outside the scope of this paper but is worthy of reference for future designers. Further data is presented in Seidel and Collingwood (2001) where examples of roughness with rock UCS is presented as a first pass estimate. Whilst less common in practice, direct measurement of socket roughness with a profiling tool is also possible.

4. The final consideration is associated with rock strength as represented by the intact strength (UCS) and fissure strength (friction). Constant volume fissure strength (φ'_{cv}) is used to assess the pile capacity when combined with socket roughness and the normal effective stress acting on the pile/rock interface as a function of in-situ stress (small effect) and stresses mobilised due to attempted dilation at the socket to rock interface. For high strength rocks the limiting strength may be that of the concrete rather than the calculated (frictional) socket shear strength.

Seidel and Collingwood (2001) took the above considerations and combined them in a dimensionless Shaft Resistance Coefficient (SRC) that can be used to predict shear stress mobilised at various displacements. The SRC is defined as:

$$SRC = \eta_c \frac{n}{1+\nu} \cdot \frac{\Delta r}{2r_s} \tag{3.16}$$

η_c construction method reduction factor as in Table 3.1.
n modular ratio (Rock Mass Modulus / rock UCS) = E_m / σ_{ci}
Δr mean roughness height of the rock socket, measured directly or assessed from the asperity length and mean asperity angle
r_s rock socket radius

The value of SRC can then be used to assess the appropriate value of α_q thus allowing for rock socket adhesion to be assessed from Equation 3.12 with allowance for cleanliness and construction, pile diameter, rock stiffness and rock socket roughness. Figure 3.6 shows the computed values for α_q with SRC. Further extensions of the work allow for assessment of mobilised resistance versus displacement to build up a full picture of pile loading.

The above comments have addressed limiting conditions in the rock. It is also necessary to check the strength of the reinforced concrete pile element in axial, bending and shear modes as well as the limiting interface strength in the concrete adjacent to the rock socket. Such considerations require consultation with the appropriate structural design codes.

Base resistance:

Base resistance may in rock sockets may be included in design when it can be demonstrated that cleanliness of the base can be achieved and where the drilling and cleaning processes have not altered the quality of the rock immediately below the socket base. When this is the case then rock socket base resistance can be expressed as a function of the unconfined compressive strength of the rock, σ_{ci}, with an equivalent bearing capacity factor as a function of rock quality and jointing. For massive or tightly jointed rock (clean joints) with a socket depth at least one diameter into equivalent quality rock, Rowe and Armitage, (1987) suggest limiting ultimate base resistance, q_b, to:

$$q_b = 2.5 \ _{ci} \tag{3.17}$$

Table 3.1 Construction method reduction factors (after Seidel and Collingwood, 2001). Note *1: Care is required to ensure that the bentonite or polymer usage is in keeping with manufacturer's specification and that the product is appropriate for ground conditions and usage for forming rock sockets.

Construction method	c
Construction without drilling fluid:	
• Best practice construction and high level of construction control (e.g. socket side wall free of smear and remoulded rock)	1.0
• Poor construction practice or low quality construction control (e.g. smear or remoulded rock present on side walls)	0.3-0.6
Construction under bentonite slurry *1	
• Best practice construction and high level of construction control	0.7-0.9
• Poor construction practice or low quality construction control	0.3-0.6
Construction under polymer slurry *1	
• Best practice construction and high level of construction control	0.9-1.0
• Poor construction practice or low quality construction control	0.8

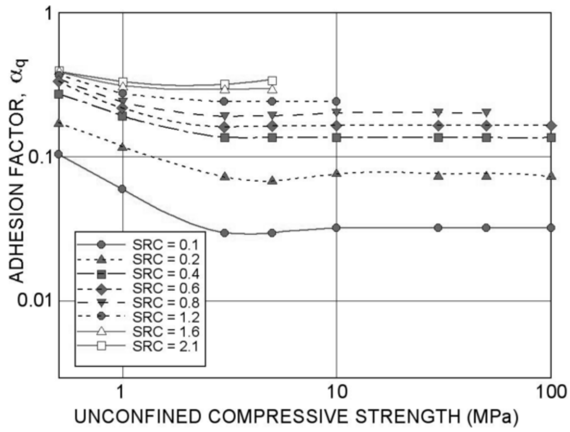


Figure 3.6 SRC and adhesion factor (α_q) (after Seidel and Collingwood, 2001)

Where rock socket embedment is less than 1 diameter, or where the rock joints are not tight and clean, reduction in capacity is required. Where rock is of lesser quality it is necessary to accommodate the rock quality in assessment of base resistance. For this case attention to the nature of the rock jointing and infill is required; the reader is referred to publications such as Rock Engineering (1994) by the US Army Corps of Engineers where a process based on rock mass considerations is provided. Where possible load testing should be considered and to this end use of Osterberg cells provides a useful tool for testing components of base and shaft resistance independently.

3.3 *Pile design – lateral loading*

A literature review for papers and publications on laterally loaded piles does not reveal the same levels of development as seen above for axial loading of piles. Recent distillations of design guidance for lateral loading include:

- GEO (2006): assessment of single piles in soil with recourse to Broms (1964) and Brinch Hansen (1961);
- Turner (2006): assessment of single piles in rock with highway structure loading, recourse to Reese (1997);
- API (2000); and
- Randolph et al (2005).

Such documents provide useful summaries of current design practice but do not reveal significant development in design processes. Turner (2006) and API (2000) both present analysis methods utilising "p-y" curves to model the soil support to laterally loaded piles. These p-y (force – displacement) springs have limiting values (p_{ur}) which can be calculated according to classical solutions such as that proposed by Brinch Hansen (1961). For limiting values of lateral resistance, Martin and Randolph (2006) report more recent work for deeply embedded piles in clay soils (undrained behaviour only) while Reece (1997) provides a calculation for ultimate lateral resistance for piles in rock as follows:

$$p_{ur} = {}_{r}\,{}_{ci}\, D\,(1 + 1.4\, x_r / D) \bullet 5.2\, {}_{r}\,{}_{ci}\, D \qquad (3.18)$$

where

${}_r$ is a reduction factor and is equal to 0.33 for an RQD of 100% increasing linearly to 1.0 for an RQD of 0%. Values of ${}_r$ above 0.5 should be used with caution and it is recommended that a limit on ${}_r$ of 0.5 is used where comparable load test data is unavailable.

x_r is depth below top of rock

Published data for p-y curves are generally for use with single piles; they do not accommodate behaviour of piles in groups where there may be significant interaction between piles as well as structural restraint at the pile head influenced by pile cap stiffness. Hence, for pile groups recourse to p-y curves assessed from single pile load test data is not acceptable for detailed design and alternatives are required. Alternative approaches include the following:

- Preliminary design can be carried out by using p-y curves to generate single pile behaviour. The results from the single pile model can then be used to calibrate pile group computer codes such as PIGLET (User manual: Randolph 2003b) with single or very widely spaced piles. Such calibration is particularly important with layered soil where the relatively simple input to pile group programs (linearly increasing stiffness with depth) does not lend itself to the stepwise varying stiffness values of layered soils. The resulting assessment of spring data can then be used for assessment of pile group behaviour. Upper and lower bounds of input parameters should be obtained for the single pile to allow pile group behaviour to be investigated.
- For detailed design, especially where there is no case history data or where pile cap interaction with the ground is important, use of 3D finite element (or similar) computer programs will provide the optimum analysis. It is however noted that offshore industry practice (e.g. API, 2000) usually reverts to methods using p-y and t-z/q-z (vertical stiffness) for pile group analysis, such methods are included in commercial computer programs with varying degrees of complexity (structural stiffness of pile cap, non-linear springs etc). These codes do not provide full soil structure interaction and as such require a degree of experience in their use for large groups of closely spaced piles is required.

3.4 *Developments in pile design – environmental considerations.*

What constitutes good design? Historically a good design was one that provided the required function at minimum cost, was constructed within the programme requirements and resulted in minimum disruption to third parties, these issues are addressed in Section 2 in terms of code requirements. The question is "can we do better than this?" Recently published books and papers on foundation design for buildings show two significant developments that should be considered in providing a "total" design:

- reuse of existing foundations;
- planning for new foundations to be reused; and
- incorporation of piled foundations into the building's heating and cooling systems.

3.4.1 *Foundation reuse*

Reuse of foundations is a critical consideration for urban regeneration and is well documented in Butcher et al (2006a) and papers within the 2006 conference "Reuse of Foundations for Urban Sites" edited by Butcher et al (2006b). Foundation reuse has the potential to:

- reduce cost and construction programmes;
- reduce the impact on archaeological resources;
- reduce disturbance to contaminated ground; and
- avoid disturbance to existing underground services and structures.

These benefits are in part countered by the need for:

- careful and advanced planning;
- additional testing and investigation;
- more complicated design; and
- potential increased costs in design and construction of pile caps and transfer structures.

The main tenets for foundation reuse are presented below and may be traced from Butcher et al (2006).

Reuse of existing foundations:

- Foundation reuse is not a decision that a designer can take alone. The decision to reuse, while laudable, must be agreed upon by the client, the client's insurers and the building control authorities (City engineer or similar). In specific cases, other interested parties may be architects and structural engineers (to ensure correct coordination of superstructure to substructure and to allow space for transfer structures), future tenants of the structure and their insurers, the list is long. The geotechnical designer can lead this process by documenting the cost, programme and environmental risks and benefits of foundation reuse thereby achieving sign-on from those parties who will be most influential in a successful outcome. The process adopted by the designer which results in pile reuse should be one where reasonable skill and care is demonstrated and this should be part of the designer's contract. It would be impossible for a designer to provide a fit for purpose guarantee for pre-existing structures regardless of the diligence of his investigation and design. As such ultimate risk, accepting reasonable skill and care by the designer will reside with the client and, hopefully, his insurers in the final place.
- Foundation reuse should not detrimentally impact on the performance of the structure being placed on it, nor should it limit the function of the new structure. If there is reasonable doubt relating to the performance of a foundation when reused then these doubts should be addressed by means of measurement or redesign. All risks and contingency plans must be agreed by risk holders before implementation of a construction reliant on foundation reuse.
- When reloaded, foundations will be stiffer than on initial loading and probably stiffer than newly built foundations. However, when foundations are loaded beyond historic maximum loads there will be a marked non-linearity in the load-settlement curve when compared to newly built piles which, while less stiff, will have a more uniform stiffness up to working load (less non-linearity). In the case of a foundation solution using both new and existing foundations the difference in stiffnesses during loading should be investigated to prevent structural over-load of piles or other elements of the sub-structure as a result of load redistribution.
- At the limit, the ultimate load capacity of old piles, especially those which have been loaded for significant periods, is seen to be larger than similarly dimensioned new piles. This potential benefit must be balanced against non-linear settlement impacts as described above.

- Structural performance of the reused foundations should be fully understood. As-built records of these foundations are valuable but must be confirmed in terms of geometry (pile diameter, length, reinforcement quantities and depths) as well as material strengths and durability. Where there are no as-built records, an enhanced level of testing will be needed. It may be that insufficient confidence can be obtained related to the existing foundations to allow reuse. The existing structure above foundations planned for reuse should be checked for distress (did the foundations perform as would be anticipated?); load intensity (what load has been applied to the piles?); and geometry (does the pile layout match the substructure / superstructure?). If these investigations suggest a building that is at odds with the anticipated foundation then further investigation will be required.
- When foundation reuse is to be adopted the existing structure must be sympathetic to foundation reuse. Access to piles must be possible to check location, geometry and materials. Can load testing be carried out using the mass of the existing building as reaction? Does demolition cause damage to the structure of existing piles (ground movement causing cracking of piles in bending or tension)?

The above list provides the basic considerations for reuse of existing foundations. The following list presents the basic requirements that present day designers and contractors should satisfy in order that reuse of new foundations is a viable option in the future. Planning for reuse:

- Provision of documentation for the future is valuable and a relatively easy task. A "close-out report" should include: full details of ground conditions, foundation design calculations, design and as-built layout and geometry (dimensions and depth) and materials; full records of testing (both pile load testing and material testing) as well as pile installation logs and non-conformance reports (with any mitigations recorded). The designer's/contractor's programme and costs should allow both the time and resources to complete such a report; without such financial consideration a "close-out" report will not be provided in a commercial world! In time the report will have a value way in excess of the cost of production. The report should be deposited with the building records maintained by the owner. Other documents to be preserved include the factual ground investigation and design reports as well as the piling specification and contractor's method statements.
- Where possible, monitoring of building performance is a useful demonstration of foundation performance; this is usually only viable for larger structures. Such monitoring includes settlement monitoring in its simplest form but may also include load monitoring in piles (strain measurements usually) or pressure monitoring below rafts. The monitoring data will validate the design and construction to working conditions and will provide subsequent owners with data to assess future foundation performance.

3.4.2 *Energy piles*

Energy piles are conventional load bearing piles which have been equipped with ducting to carry a circulating thermal fluid, usually attached to the reinforcement cage as illustrated in Figure 3.7. The ducting allows the piles to be used as heat exchange elements as well as being load carrying structures. The thermal fluid in the ducting is usually allied to heat pumps to amplify temperature differences. This allows the energy piles to be used as part of the building management system (BMS) by providing all or part of the heating / cooling demand of the building.

The use of energy piles has been widespread in continental Europe since the 1980's. Elsewhere, it is gaining importance as a means of improving the green credentials of new build structures.

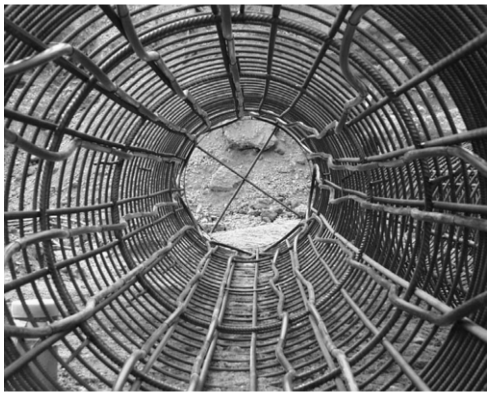


Figure 3.7 Pile with tubing installed prior to pile construction. The tubing is often high density polythene plastic pipes of 20 or 25mm diameter. The circulating fluid typical comprises either water, water and an anti-freeze solution or a saline solution depending on the operational temperature range

Brandl (2006) summarises the current considerations for energy pile design in a detailed presentation of his 2001 Rankine Lecture, illustrating good practice by means of case history data and analysis. Energy piles have the ability be used cyclically by extracting heat from the ground when the building is cooler than required and providing "coolth" when the building is warmer than required. The "energy" part of energy piles is usually subservient to their load carrying capacity, so care to prevent the cyclic cooling and heating of the piles and surrounding ground from causing damage to the soil matrix and pile structure is a prerequisite. Brandl describes the process where the heating-cooling cycle is annual and this is the most typical requirement for domestic situations. It is however noted that the heating-cooling cycle can be daily allowing significantly higher temperature differentials to be achieved. Such systems require more sophisticated management and are more appropriate to commercial buildings where peak daily conditions require cooling during day time and heating during night time (e.g. office buildings with significant computer usage or retail with high energy lighting). Where energy piles are operated with conductive fluids at less than 0°C, greater care in the design will be needed to ensure that temperatures less than 0°C are restricted to the pile structure and do not reach the ground. In such situations accurate modelling of thermal properties of materials and the construction geometry of a the piles are important; Figure 3.8 shows a model simulation of an energy pile after 15 hours operation with a circulating fluid at -5°C. It is clear that the pile edge temperature has dropped to 1°C. Further operation at a temperature of -5°C would result in local, increasing to general, freezing of the ground around the pile and then potentially catastrophic thawing. The rate and duration of energy extraction from the ground should be analysed using thermodynamics to check that the ground does not freeze (and detrimentally thaw at a later date) and that the pile structure is not damaged due to temperature induced stresses. An investigation of the latter of these issues has been carried out by Bourne-Webb et al (2009) where thermal changes where induced on an energy pile during (maintained load) preliminary load testing. The pile load test demonstrated a relatively simple model associated with skin friction on the pile shaft being mobilised to resist both applied pile load and thermally induced strains. The result is modified stresses in the pile varying with depth and the need to account for these stresses in structural design of the pile. Bourne-Webb et al also note that the peak skin friction mobilised during the thermal test was marginally lower than the expected design value. Whilst the difference between mobilised and expected skin friction was not particularly large (17%) it is further evidence that the design of energy piles must consider how thermal effects will impact on static design, especially where partial factors on skin friction are low. More maintained load testing is required with thermal variations superimposed on the pile.

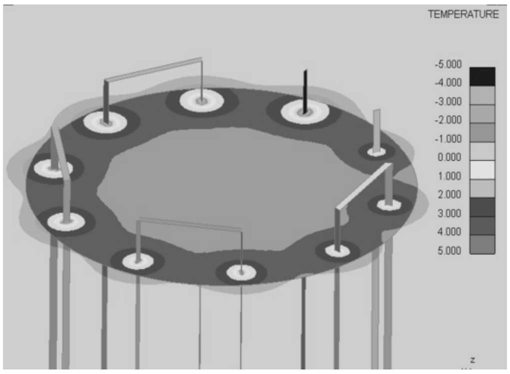


Figure 3.8 Section of a pile model with circulating fluid at -5°C after 15 hours operation.

In general, thermal piles are usually associated with conventionally bored cast-in-place piles, although driven reinforced concrete piles with integrated absorber pipes are occasionally used. CFA piling, requiring the plunging of the thermal ducting and reinforcement cage into wet concrete, is not generally advised due to the possibility of damage to the thermal pipes during cage installation into wet concrete. Nevertheless it is anticipated that development of thermal CFA piles will continue due to the economic benefits of such piles. Construction modifications, beyond that which is needed for structural considerations, include stiffening the reinforcement cage to help support and protect the thermal ducting for systems with high energy demand and the development of plunging mandrels to install the ducting below the cage in CFA piles where lower energy demands are appropriate. For details on design the reader is referred to Brandl (2006) in the first place and thereafter to 2009 issues of Géotechnique for the Symposium-in-Print on "Thermal behaviour of the ground" (in preparation at time of this article being submitted).

3.5 *Conclusions – Piling developments*

It is hoped that the above sections show how the field of pile design, analysis and construction continue to develop. Examples of offshore driven piles in sand and rock socket piles have been presented with shorter comments on axial design of piles by means of the Menard Pressuremeter. The common thread that these two examples exhibit, especially for shaft friction, is that of detailed assessment of data within a framework that is related to the model by which skin friction is mobilised. Care must be taken with extrapolating these methods to pile sizes beyond their respective data bases even though both methods include pile diameter effects.

The second topic addressed in this chapter is that of sustainable design. The need to achieve not only economic but sustainable designs is ever pressing. Such pressures should facilitate cross fertilisation of skills and knowledge from other areas of engineering as well as encouraging designers and contractors to look to the future in all that they do. The move to include more than one function in the design of a pile is laudable as are the benefits of reusing existing piles and providing documentation to encourage future reuse of current piling projects.

4 EMBANKMENTS AND SLOPES

4.1 *Introduction*

In this section of the paper, recent developments in the analysis and design of embankment sand slopes is described, with a particular emphasis on Japane≤se practice. Slope stability is a very important issue in a seismically active region, resulting in active development of both empirical and analytical methods of design.

4.2 *Basic elements of design – embankments*

4.2.1 *Classification of embankments and items to be considered in the design of cut slope*

Figure 4.1 shows the main types of embankments considered in this paper: road embankments, railway embankments, reclaimed areas for housing lots and buildings, river dikes and sea dikes. Design methods for dams are more severe and are not included here. Items to be considered in the design are: fill material, density of embankment, gradient and height of slope, and drainage facilities. In addition, seepage of water must be considered in river and sea dikes.

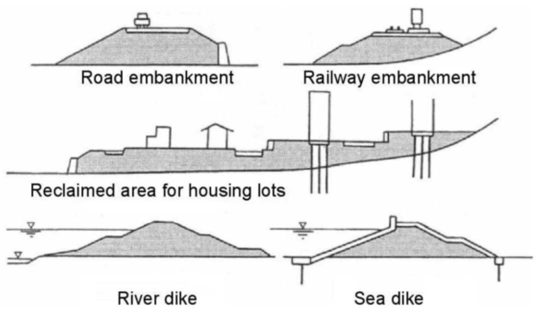


Figure 4.1: Types of Embankments

4.2.2 *Fill material*

Selection of appropriate fill material is important because bad material may cause poor workability, settlement or instability. Good fill materials have the following properties:

1. Placing and spreading, and compaction of the material is easy.
2. Shear strength and bearing capacity are high, and compressibility is low.
3. Water intake swelling is low.
4. They are stable against erosion and shear strength does not decrease due to saturation.

Well graded gravelly or sandy soils satisfy these conditions and are recommended for use as fill material. On the contrary, expansive soils such as bentonitic or solfataric soil (sulphurous volcanic soil) and highly organic soil cannot be used to construct embankments. Attention is necessary to the following special soils:

a. Volcanic cohesive soil: clay originating from volcanic ash. Void ratio and natural water content are very high and unit weight is very low. Therefore, trafficability is low, and the embankment may suffer from slope failure or long-term settlement.
b. Some sedimentary soft rocks such as mudstone, shale and tuff: these may suffer slaking and water intake swelling after filling.

4.2.3 *Density of embankments*

Appropriate placing and spreading, and adequate compaction must be conducted during construction. In general, the compaction control standard is decided before construction. Several compaction control methods are available as follows:

a. Compaction control by density based on degree of compaction.
b. Compaction control by degree of saturation.
c. Compaction control by strength of deformation.
d. Compaction control by compaction method.

4.2.4 *Gradient and height of slope*

In general, failure of an embankment occurs for one of two main reasons: a) failure due to insufficient strength of the foundation ground during construction of the embankment, and b) failure triggered by heavy rains or earthquakes after construction. In the case of dams, their height and inclination of slope are evaluated by conducting slope stability analyses. However, in case of road and railway embankments, inclinations of their slopes are usually designed empirically without conducting slope stability analyses, because their length are very long. In the empirical approach, the appropriate inclination of a slope is decided by the height and the material of the embankment. The empirical design method is not uniform throughout the world because fill and ground conditions are quite different in each country. For example, road embankments are designed based on Table 4.1 in Japan.

Table 4.1 Empirical design method for road embankment in Japan (Japan Road Association, 1999)

Fill Material	Height of embankment	Inclination of slope
Well graded sand, Gravel, Gravel with fines	Less than 5m	1:1.5 to 1:1.8
	5m to 15 m	1.1.8 to 1:2.0
Poor graded sand	Less than 10m	1.1.8 to 1:2.0
Rock, Muck	Less than 10m	1.1.5 to 1:1.8
	10m to 20m	1.1.5 to 1:1.8
Sandy soil, Hard clayey soil, Hard clay	Less than 5m	1.1.5 to 1:1.8
	5m to 10m	1.1.8 to 1:2.0
Volcanic clay	Less than 5m	1.1.8 to 1:2.0

If the embankment is high and/or the material is soft, low inclination is selected. However, slope stability analysis is necessary if conditions for the foundation ground and/or the embankment are bad as follows:

a. the underlying ground is soft.
b. the embankment is constructed in landslide area.
c) the height of the embankment is greater than the heights listed in Table 4.1.
d. the fill material is bad, such as high moisture clay or volcanic ash.
e. houses exist near the embankment that could be damaged if the embankment is deformed.

For stability, circular slip surface analysis is widely used. However, compound slip surface analysis must be conducted if a thin soft layer exists beneath the embankment or if the surface of the foundation ground is inclined. Procedures to estimate safety factor by finite element methods have also been developed recently.

In seismic design, the seismic force is added in the stability analysis. Moreover, the effect of excess pore water pressure is considered if the foundation soil and/or fill material are liquefiable. One formula considering the excess pore-water and derived based of Fellenius's method is as follows:

$$F_s = \frac{\sum [cl + \{(W - ub)\cos\alpha - k_h W \sin\alpha\}\tan\phi]}{\sum\left(W \sin\alpha + \frac{h}{r} k_h W\right)} \quad (4.1)$$

in which u is the pore water pressure including excess pore water pressure due to liquefaction, and k_h: is the horizontal seismic coefficient.

Fellenius' method is used for simplicity, although its shortcomings are acknowledged (Lambe & Whitman 1979). One simple method to estimate excess pore-water pressure due to liquefaction (Japan Road Association, 1986) is to assume that R_u is 1.0 when $F_L \leq 1.0$, where, R_u is the ratio of excess pore-water to effective overburden pressure. When R_u = 1.0, the entire weight of overburden is taken on the water pressure.

A new design concept to introduce performance-based design (PBD) has been developed recently. In the PBD, deformation of an embankment must be evaluated. Several methods studied recently are:

1. Dynamic response analysis (FEM)
2. Static residual deformation analysis (FEM)
3. Newmark's method (Newmark 1959)

4.2.5 *Drainage facilities*

Drainage is very important to construct safely and maintain stable embankments. Usually seepage water and surface water must be cut off from surrounding area into the construction site temporarily, during the construction of embankments. In the design of permanent drainage, two forms of drainage must be considered as shown in Figure 4.2: (a) drains for surface water, including subsurface drainage and slope surface drainage, and (b) facilities against seepage water into the embankments, such as horizontal drains, horizontal blankets, drainage pipes or gabions.

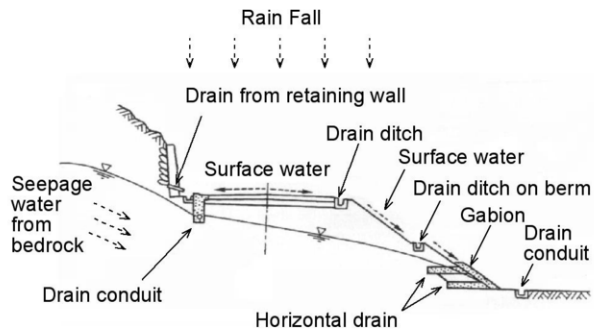


Figure 4.2: Drain facilities for embankments

Special care for drainage is necessary to the following locations:

a. Small rivers or boundaries between cuts and embankments where flows of surface water concentrate
b. Valleys, and boundaries between cuts and embankments where much spring water emerges.

4.3 *Basic elements of design- Cut slopes*

4.3.1 *Items to be considered in the design of cut slopes*

In the design of cut slopes, slope gradient, drainage facilities, slope protection works and maintenance works must be evaluated.

4.3.2 *Design of slope gradient*

If the natural ground of the slope is composed by collapsible soils such as shown in Table 4.2, soil investigation, soil tests and slope stability analyses are necessary to design appropriate slope gradients. If there is a flow of water in the ground, slope stability analysis is necessary even in normal soils.On the contrary, if the slope is protected from water flow, the gradient of the slopes composed by normal soils is often designed by empirical methods without conducting soil investigation, soil tests and analyses. In the empirical methods several factors, such as hardness of soils, height of cut slope, are considered. The empirical design method is not uniform throughout the world because soil conditions of slopes are quite different in each country. For example, road cut slopes are designed based on Table 4.3 in Japan.

Table 4.2 Collapsible soils

Soil type	Typical soils
Weak for erosion	Welded tuff, weathered granite
Unconsolidated or weathered	Talus, Volcanic soil, Colluvial deposit
Weathering speed is high	Mudstone, Tuff, Shale, Slate, Serpentine
Fissured	Shale, Serpentine, Granite, Andesite
Fissures forms dip slope	Slate
Sandwiching soft layers	Fault clay, Post landslide or collapsed site

Table 4.3 Empirical design method for road cut slopes in Japan (Japan Road Association, 1999)

Soil of cut slope		Height of cut slope	Inclination of cut slope
Hard rock			1:0.3 to 1:0.8
Soft rock			1:0.5 to 1:1.2
Sand	Not dense and poor graded		More than 1:1.5
Sandy soil	Dense	Less than 5m	1:0.8 to 1:1.0
		5m to 10m	1:1.0 to 1:1.2
	Not dense	Less than 5m	1:1.0 to 1:1.2
		5m to 10m	1:1.2 to 1:1.5
Gravelly sand	Dense or well graded	Less than 10m	1:0.8 to 1:1.0
		10m to 15m	1:1.0 to 1:1.2
	Not dense or well graded	Less than 10m	1:1.0 to 1:1.2
		10m to 15m	1:1.2 to 1:1.5
Clay		Less than 10m	1:0.8 to 1:1.2
Gravelly clay		Less than 5m	1:1.0 to 1:1.2
		5m to 10m	1:1.2 to 1:1.5

For slope stability, compound slip surface analyses or circular slip surface analyses are commonly used for shallow slides and deep slides, respectively. In seismic design, seismic force is added in the stability analysis. However, this is seldom to be conducted. In the new design concept to introduce performance-based design, deformation of cut slope may be evaluated by dynamic response analysis or Newmark's method. But more studies are necessary to introduce these methods.

4.3.3 *Drainage facilities*

For cut slopes, several kinds of facilities must be equipped for drainage as schematically shown in Figure.4.3. These drains are classified into two groups: (a) facilities against surface water, such as catch ditches at the tops of slopes, drain ditches on berms and drain channels along slopes, and (b) facilities against seepage water and spring water, such as drain conduits at the toes of slopes, horizontal drainage pipes and drainage wells.

4.3.4 *Slope protection works against sliding, weathering or erosion*

Slope protection works are classified into two groups: planting treatment and slope protection by concrete pitching, reinforcing grids, sprayed concrete etc. The latter is applied only to the following conditions because of high cost and bad appearance.

a. The soil of the slope is too hard or acid for vegetation.
b. The slope is unstable against sliding.
c. Protection against erosion is necessary because of running surface water from rainfall or springs causing slope failure.

The planting treatment is conducted by spraying of seeds, pasting of special mats which are processed by soils and seeds, or turf work. In the slope protection by concrete pitching etc, four types of methods are in use:

a. protection against weathering and erosion: guniting (spaying mortar), shotcreting (spraying concrete), stone facing,
b. protection against flow of soils due to erosion and spring water: gabion
c. protection against sliding: concrete pitching, concrete crib work.

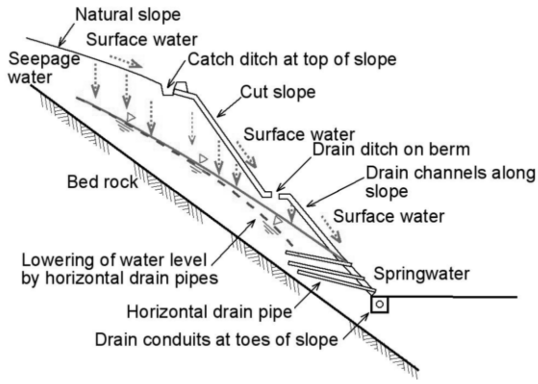


Figure 4.3: Drainage facilities for cut slope

4.3.5 *Retaining walls*

If the cut slope is steeper than the stable inclination, construction of a retaining wall at the toe of the slope is appropriate. If the wall is high, such as more than 5m, stability of the wall against earth pressure must be checked by calculation; for lower walls, empirical design based on experience may be used.

4.3.6 *Maintenance*

Ground and protection work of cut slopes becomes deteriorate with age owing to several factors. The most important factor is rainfalls and seepage from bedrock, for which appropriate treatment is necessary based on periodical site inspection of seepage water and drainage conditions.

4.4 *Basic elements of design - natural slopes*

4.4.1 *Classification of landslides*

There are many types of landslides of natural slopes. If the landslides are classified by speed of sides, landslides are classified into two groups:

1. Rapid landslides
   Steep slopes slide rapidly during heavy rainfalls or earthquakes. Slipped masses fall down to the foot of slopes. Surface slides of weathered soils are dominant, but if some deep layers are week, deep slides may occur during earthquakes. For example, the maximum depth of the slip surface of Ontake Slide which occurred during the 1984 Naganoken-seibu earthquake was about 180m. Stability of natural slopes is generally judged roughly by several factors such as angle of slope, existence of unstable rocks, histories of slides. Stability analyses are conducted in special cases only. In the analyses, stability of the slides can be evaluated based on peak strength of the soils. However, prediction of unstable slopes over wide areas during future rains and earthquakes is not easy.
2. Slow landslide (land creep)
   Gentle slopes may creep slowly and periodically due to seasonal change of water level. Sliding may occur particularly in spring if the slope is affected by melting

snow, possibly sliding a few metres each year on weak layers, such as weathered pumice and sandstone. Stability of the slide can be evaluated by slope stability analyses based on residual strength. As the slides occur periodically, slide prone sites can be identified easily, and countermeasures can be applied.

4.4.2 *Countermeasures*

Two types of countermeasures against landslides may be introduced: control works and prevention works. In control works, the stability of a slope can be increased by controlling ground water level or the shape of the slope. In prevention works, failure of the slopes is prevented by constructing some countermeasures against sliding. Some countermeasures are listed in Table 4.4.

Table 4.4 Countermeasures against landslides

Type of countermeasure	**Countermeasure methods**
Control works	Surface water drainage work Groundwater drainage work • Ground water cut-off wall • Soil removal work • Counterweight fill method
Preventing works	Retaining wall Soil reinforcement Ground anchorage Pile works, Shaft work

4.5 *Recent developments- embankments*

4.5.1 *Effective use of construction generated soils*

Recently reconstruction of buildings and new construction of underground structures have increased in urban areas. Consequently, the availability has increased of soils from excavations and broken brick and concrete from demolition of buildings. These materials are difficult to dispose of because of lack of landfill. So it becomes necessary to use these materials for fill material or other purposes. Often, excavated soils are not suitable for fill materials because their water content is too high and/or they are unsuitable mixtures of coarse and fine grains. Then special treatments are conducted as follows:

a. lowering of water content by special bags or sandwich methods;
b. treatment of soil with air foam, fibers, cement or lime.

4.5.2 *Lightweight embankments*

A recently developed method to prevent large settlement due to construction of embankments on very soft ground is to use light weight material. EPS (Expanded polyestyrene) construction block, lightweight treated soil with air foam (see, for example, Tsuchida et al 2001), lightweight treated soil with EPS beads, or fly ash may be used as the light weight material. The use of EPS method has become especially popular recently. In this method, EPS blocks are stacked and covered by soils. If the weight of the soils for cover is heavy, settlement at the toe of embankment occurs as schematically shown in Figure 4.4. Therefore attention is necessary not only the weight of EPS but also that of the covered soils. Floating due to buoyancy and disturbance due to strong wind must be prevented during construction. Moreover, EPS may be burned if there is a fire due a to traffic accident, and it can also be damaged by petrol spillage.

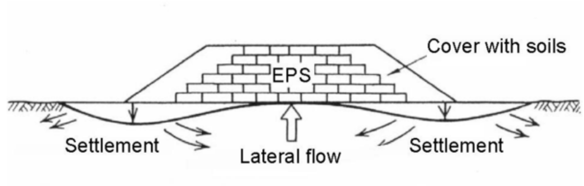


Figure 4.4: Deformation of an embankment constructed by EPS method due to the weight of covered soil

4.5.3 *Judgment of instability of embankments by field observation*

If the ground is very soft, stability of an embankment is evaluated not only before construction but also during the construction of the embankment. Settlement and spreading of the toe of the embankment are measured and may be plotted on a control diagram such as Figure 4.5.

If the plotted point exceeds a failure (critical) line, it is judged that the embankment is becoming unstable, and the construction cannot be continued. An appropriate measure, such as waiting several days or constructing counterweight fill, is applied. Many control diagrams have been proposed based on case studies of failed embankments.

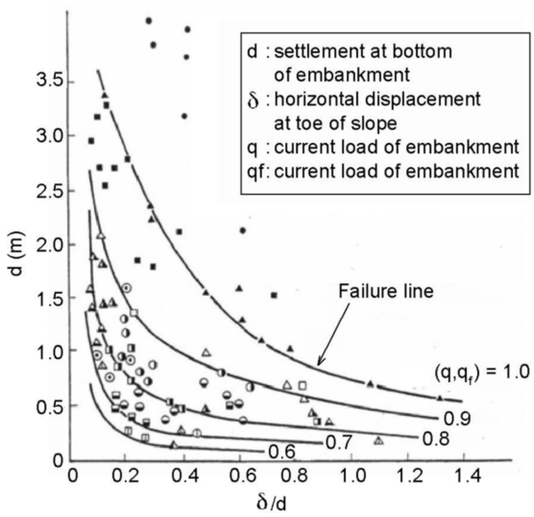


Figure 4.5: Diagram to judge instability of embankments (proposed by Matsuo and Kawamura 1977)

4.5.4 *Introduction of performance-based design in seismic design*

The need to evaluate not only the safety against sliding but also deformation of embankments was recognized after the 1995 Kobe earthquake. In river dikes, the aim is to protect from flooding, so the critical requirement of the dikes is prevent overflow of river water, as schematically shown in Figure 4.4(a). In road embankments, emergency vehicles must run just after earthquakes. For example differential settlement of approaches to bridges from embankments must be within the appropriate value for the vehicles, as shown in Figure 4.4(b). Thus, differential settlement must be one of the critical conditions in design of road embankments. One more reason why the evaluation of deformation of embankments must be introduced in seismic design, is that the calculated safety factor against sliding, F_s, is apt to be lower than 1.0 under the Level 2 earthquake motion, even though the ground is medium dense. Therefore F_s cannot be used in the design under Level 2 shaking. Here, Level 2 shaking motion is defined as the maximum shaking motion. Level 1 shaking motion is defined as

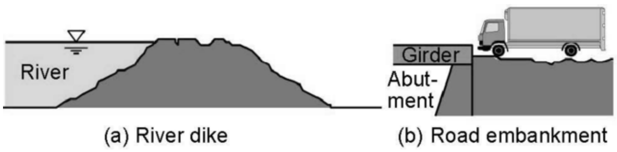


Figure 4.6: Critical conditions for river and road embankments

the shaking motion which occurs two or three times during the lifetime of a structure Recently, allowable settlements for super levees (Japanese River Association, 1997), railway embankments (Railway Technical Institute, 1999) and river levees (River Bureau, 1995) have been introduced in Japanese design manuals and guidelines. A "super levee" is an extended area of raised ground behind the main levee, on which construction may take place as shown in Figure 4.7. In the manual for super levees, allowable settlements are 50 cm for the top of the levee and face of the back slope, and 20 cm for the ground on the super levee. As the super levees are used for residential areas, similar safety as urban areas is necessary. Hence these allowable values were introduced in the design manual.

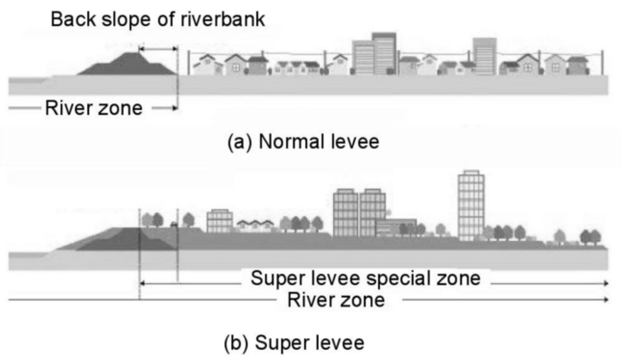


Figure 4.7: Difference between normal and super levees

In the guideline for railways, damage levels of deformation of embankments are classified into four grades as shown in Table 4.5 and Table 4.6 for embankments and approaches to bridges, respectively. If the settlement is more than 50 cm it can be judged that long term remedial work is necessary. On the contrary, it can be judged that the damage is slight if the settlement is less than 20 cm. Therefore allowable settlements under Level 1 and 2 earthquake motions were decided as 20 cm and 50 cm, respectively as shown in Table 4.5. Recently, seismic diagnosis of existing river dikes has been conducted in Japan. In the diagnosis, allowable settlement is defined as shown in Figure 4.8. It is recommended that the level of a river crest after an earthquake must be higher than the level of the mean monthly highest water plus wave height.

In the estimation of liquefaction-induced deformation of structures, three types of methods are available: empirical methods, static analyses and dynamic analyses. One empirical method is introduced in the design manual for river dikes, as shown in Table 4.7. In this method, settlement of a dike is estimated by the safety factor of the slope, F_s. Two values of F_s must be calculated: F_s (k_h) which considers the seismic coefficient, and $F_s(\Delta u)$ which considers excess pore water pressure due to liquefaction. Then the settlement is estimated by the lower F_S. The relationship shown in Table 4.7 was derived from the correlation between settlement of damaged dikes and F_s during several past earthquakes as shown in Figure 4.9.

Table 4.5: Damage level for railway embankment (Railway Technical Institute 1999)

Deformation level	Damage level	Settlement, S (roughly speaking)
1	No damage	None
2	Slight damage	S<20cm
3	Medium damage (restoration is available with emergency repairs)	20cm≦S <50cm
4	Severe damage (long term restoration is necessary)	S≧50cm

Table 4.6: Damage level for differential settlement between abutment and embankment (Railway Technical Institute 1999)

Deform-ation level	Damage level	Differential settlement between abutment and embankment, S_d (roughly speaking)
1	No damage	None
2	Slight damage	S_d<10cm
3	Medium damage (restoration is available with emergency repairs)	10cm≦S_d<20cm
4	Severe damage (long term restoration is necessary)	S_d≧20cm

Table 4.7: Relationship between F_s and settlement (Public Works Research Institute 1997)

Safety factor of slope, F_s		Settlement (maximum)
$F_{sd}(k_h)$	$F_{sd}(\Delta u)$	
1.0<F_{sd}		0
0.8<F_{sd}<1.0		0.25H
F_{sd}0.8	0.6<F_{sd}<0.8	0.5H
	F_{sd}<0.6	0.75H

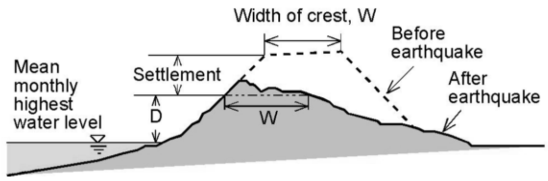


Figure 4.8: Definition of allowable settlement for river dike (River Bureau 2007).

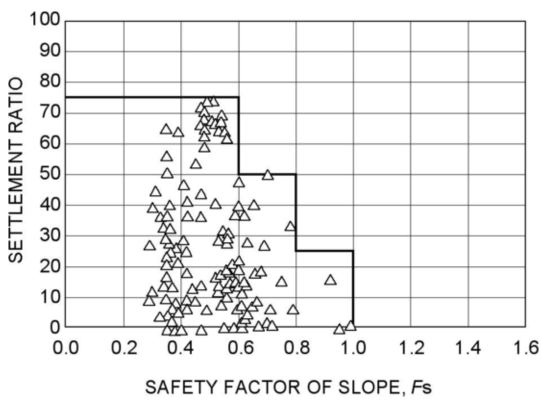


Figure 4.9: Relationship between F_s and settlement ratio of river dike (Public Works Research Institute, 1997)

Figure 4.10 shows another empirical relationship between settlement of dikes of Kiso, Nagara and Ibi Rivers during the 1944 Tohnankai earthquake, and liquefaction potential, P_L at the damaged sites. As show in this figure, the settlement increased with the value of P_L. P_L is calculated by the following formula together with Figure 4.11 (Iwasaki et al., 1978).

$$P_L = \int_0^{20} (1 - F_L)(10 - 0.5z)dz \tag{4.2}$$

where F_L is the safety factor against liquefaction, equal to the ratio of undrained cyclic shear strength to cyclic shear stress. The term $(1\text{-}F_L)$ is set to 0.0 if F_L>1.0

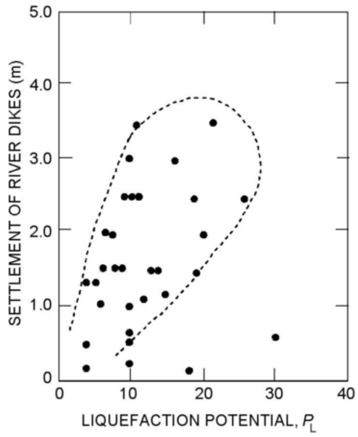


Figure 4.10: Relationship between P_L and settlement of river dikes (Nakamura and Murakami 1980)

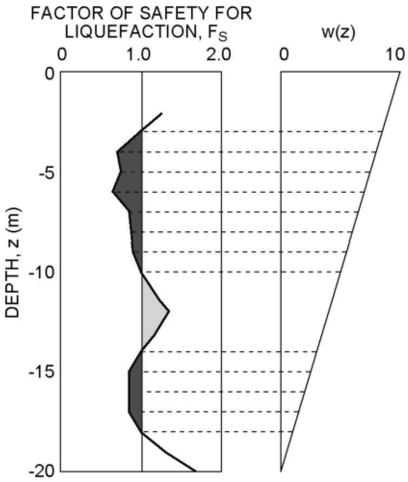


Figure 4.11: Definition of P_L (Iwasaki et al 1978)

For railway embankments, a relationship among settlement, height of embankments, density, number of cycles and liquefaction potential, P_L is prepared to estimate the settlement of embankments as shown in Figure 4.12. This relationship was derived from shaking table tests.

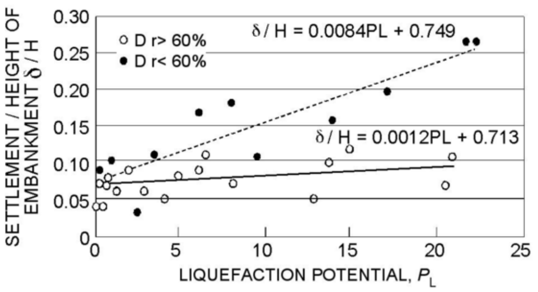


Figure 4.12: Relationship between P_L and settlement ratio for railway embankments (Sawada et al 1999)

In dynamic and static analyses, several methods have been proposed and applied to estimate the deformation of embankments. A technical committee organized by the Japanese Institute of Construction examined the efficiency of these analytical approaches. In the examination, two dynamic methods of analysis, LIQCA and FLIP, and two static methods, ALID and Towhata's method, were applied to seven actual river dikes which were damaged and non-damaged during the 1993 Hokkaidonansei-oki earthquake and 1995 Hyogoken-nambu earthquake. LIQCA and FLIP are two-dimensional effective stress computer codes developed by Oka et al. (1999) and Iai et al. (1992), respectively. ALID is a simplified method using static FEM developed by Yasuda et al. (2003) by assuming that residual deformation would occur in liquefied ground due to the reduction of shear modulus. Towhata's method was developed based on a minimum energy principle. Figure 4.13 shows the comparison between the calculate dike settlements and the observed settlements. Settlements estimated by the empirical approach shown in Table 4.7 are also compared in the figure. The predicted settlements by the analytical approaches agree fairly well. Figure 4.14 shows the analysed deformation by ALID at a severely settled dike. These methods were also applied to models with countermeasures tested on a shaking table apparatus, to demonstrate the applicability of the analytical methods to the dikes with countermeasures.

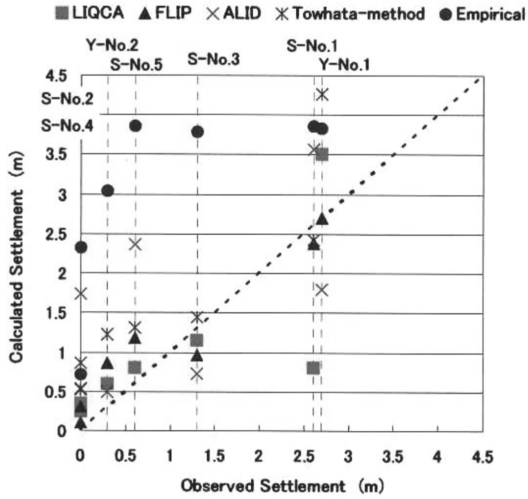


Figure 4.13: Comparison between calculated and observed settlements (Sasaki et al 2004)

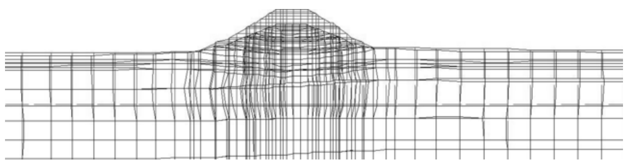


Figure 4.14: Analysed deformation by ALID (Shiribeshi-toshibetsu River, No.1) [8]. Height of river dike: 5.3m. Settlement: 2.3 m. (Yasuda et al 2003)

4.5.5 *Development of strengthening techniques of existing embankments (partially quoted from Yasuda (2007)*

If the surface soil of the foundation ground is very soft clay or peat, soil improvement is necessary before construction of the new embankment. Many improvement techniques have been developed such as sand drains, paper drains, preloading, and cement-mixing methods. Recently, it has become necessary to develop soil improvement techniques for existing embankments. In particular, it is desired to develop appropriate countermeasures for the embankments on sandy ground which is liquefiable due to earthquakes.

If the soil of an embankment liquefies, it is necessary to prevent the flow of the liquefied soil. Lowering the water level in the embankment by installing horizontal pipe drains may prevent liquefaction. In contrast, if the soil under an embankment liquefies, large settlement of the embankment occurs due to horizontal movement of the ground below. In this case, restricting the movement by some technique, such as the installation of underground walls at the toes of the embankment, can reduce the settlement. Several techniques which have been applied to existing embankments recently are schematically shown in Figure 4.15. The Tokaido Shinkansen Railway (Japanese Bullet Train) runs over areas where liquefaction is possible during future earthquakes. As liquefaction may damage railway embankments, a sheet-pile enclosure method has been developed to protect them as illustrated in Figure 4.15(a) (Japanese Geotechnical Society 1998). The 1995 Kobe earthquake caused extensive damage to the Yodogawa dike. The deep mixing method was applied to the embankment of the Arakawa River embankment in Tokyo, as shown in Figure 4.15(b). A loose sand layer where liquefaction was anticipated was 3 to 6 m thick. It was planned to use the deep mixing method for a width of 10m and a depth of 24m to stabilise the foundation ground against external seismic forces, which are active seismic earth pressure, passive seismic earth pressure, excess pore water pressure and dynamic water pressure. Using external forces, a stability analysis was conducted that included sliding, overturning, bearing capacity of the stratum, and circular slip failure. The detailed design method of Arakawa Dike is explained in Yasuda (2007).

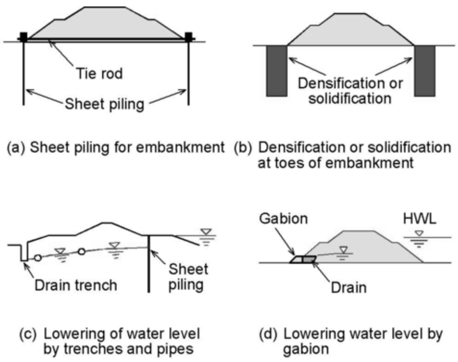


Figure 4.15: Remediation methods for existing embankments

4.5.6 *Introduction of seismic design for reclaimed area for housing lots*

Though slope failures in housing lots cause severe damage to houses and loss of lives, much reclaimed land has been developed in urban areas without considering seismic stability. Then appropriate procedures are needed to evaluate seismic stability of existing reclaimed areas. Recently, this procedure has been discussed and a guideline has been proposed by the Kanto Branch of the Japanese Geotechnical Society (2007), see also Yasuda (2007). In the guideline, it is recommended to evaluate the seismic stability by the following five steps.

Step 1: Finding out of filled zone

In a reclaimed area for a housing lot, it is difficult to find the boundary of cutting and filling by site survey, if there is no record on reclamation work. Then the guideline recommends finding the filled zoned by comparing old and new aerial photos. The filled zone can also be estimated by comparing two sets of topographical maps drawn before and after the reclamation work. However the accuracy of the estimated zone is likely to be lower than that measured by aerial photos.

Step 2: Field investigation to select unstable slopes

The next step is the identification of existing fill slopes which will become unstable during earthquakes. This can be evaluated roughly by visual inspection, based on several factors shown in Table 4.8.

Table 4.8: Factors introduced in visual inspection to evaluate seismic stability of existing fill slope

Item	Factors introduced in visual inspection
Fill	Height of fill, inclination of slope, Banking material, Type of slope protection work, Deformation of slope, Springwater, Drainage well
Retaining wall, house	Type of retaining wall, Type of weeping, Deformation of retaining wall, Springwater, Deformation of foundation of house
Topography	Catchment, Land use on the slope

Several evaluation methods based on these factors have been developed in Japan, which are introduced in the guideline. For example, a rough check sheet by visual inspection for an existing retaining wall prepared by Yokohama City is shown in Table 4.9.

Step 3: Soil investigations and laboratory tests

For the selected unstable slopes, detailed soil investigation and laboratory tests are necessary to evaluate the seismic stability of the slopes more precisely.

Step 4: Analyses on slope stability or deformation

Based on soil investigations and laboratory tests, analyses for slope stability or deformation are conducted. In the analyses of slope stability during earthquakes, seismic force and excess pore water pressure must be considered as shown in Figure 4.16. Then, in the guideline, methods to evaluate excess pore water pressure are introduced at first. One simple method to estimate excess pore-water pressure due to liquefaction the is same as mentioned above in 4.2.4 setting R_u to 1.0 for $F_L \leq 1.0$ (Japan Road Association, 1986).

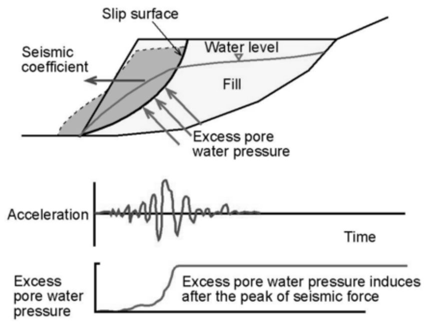


Figure 4.16: Time of the occurrence of seismic force and excess pore water pressure

As illustrated in Figure 4.16, little excess pore water pressure is usually induced before the peak of seismic motion (Ishihara and Yasuda 1975). Therefore two sets of analyses (a) to consider seismic coefficient only, and (b) to consider excess pore water pressure only, are recommended for the evaluation of seismic slope stability, as shown in Table 4.10.

In performance-based design, it is necessary to estimate not only the stability but also deformation of embankments. Several methods to estimate the deformation of embankments are introduced in the guideline. These are:

1. Dynamic response analysis
2. Newmark's method
3. Static residual deformation method

Step5: Selection of appropriate countermeasures

The final step is the selection of appropriate countermeasures against slope instability during earthquakes. Three types of countermeasures against instability of existing fill slopes are introduced in the guideline: control works, preventing works and slope protection works. Some of these are listed in Table 4.11. Examples of typical arrangements are shown in Figures 4.17 and 4.18.

Table 4.9: Rough check sheet by visual inspection for existing retaining wall prepared by Yokohama City.

		Value					
		Stone wall			RC wall		
Classific-ation	Item for check	Normal	Slightly abnormal	Abnormal	Normal	Slightly abnormal	Abnormal
Drainage condition	Drain hole	0	1.0	2.0	0	1.0	2.0
	Ground behind wall	0	1.0	2.0	0	1.0	2.0
	Percolate from wall	0	0.5	1.0	0	0.5	1.0
	Inclination of the ground behind wall	0	1.0	2.0	0	1.0	2.0
	Drainage facilities of the ground behind wall	0	1.0	2.0	0	1.0	2.0
Wall	Height and inclination of wall	0	2.0	4.0	-	-	-
	Horizontal crack of wall	0	4.0	6.5	0	3.0	5.5
	Vertical or cross crack of wall	0	2.5	5.0	0	1.5	4.0
	Crack at the corner of wall	0	3.0	5.5	0	2.0	4.5
	Horizontal movement	0	3.5	6.0	0	2.5	5.0
	Differential settlement	0	4.5	7.0	0	3.5	6.0
	Opening of the corner of wall	0	4.5	7.0	0	3.5	6.0
	Expansion of wall	0	5.0	8.0	-	-	-
	Tilt of wall	0	5.5	9.0	0	4.5	8.0
	Corrosion of iron bar	-	-	-	0	5.0	8.0
	Total value						
Judge	Total value is less than 5		Judge as less abnormal by inspection				
	Total value is greater than 5 and less than 9		Judge as abnormal by inspection				
	Total value is greater than 9		Judge as quite abnormal by inspection				

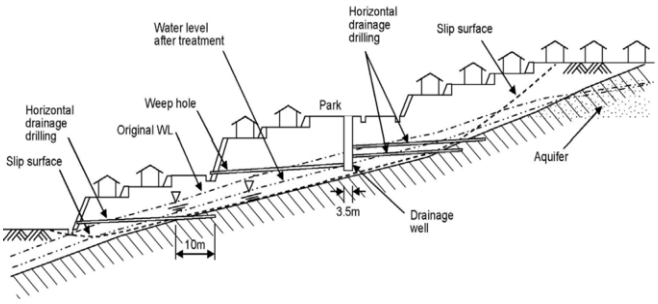


Figure 4.17: Countermeasures by horizontal drainage drilling and drainage wells (Kanto Branch, JGS, 2007)

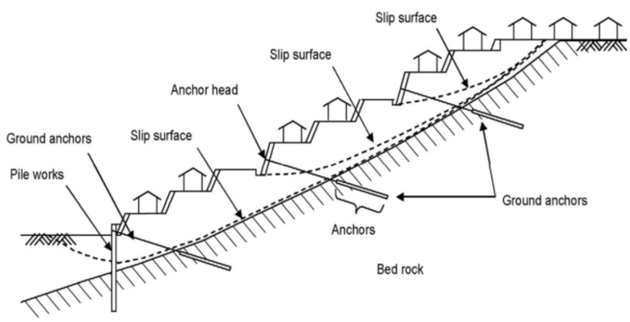


Figure 4.18: Countermeasures by ground anchors and pile work (Kanto Branch, JGS, 2007)

Table 4.10: Two factors to be considered in stability analysis.

Type of filled soil		**Factors to be considered**	
		Excess pore water pressure	**Seismic coefficient**
Sand	Loose and saturated	yes	No
	Dense and/or unsaturated	No	yes
Clay		no	yes

Table 4.11:Countermeasures against instability of existing fill slope and arrangement plans

Type of countermeasure	Countermeasure methods
Control works	Surface water drainage work, Groundwater drainage work, Ground water cut-off wall, Soil removal work, Counterweight fill method
Preventing works	Retaining wall, Soil reinforcement, Ground anchorage, Pile works, Shaft work
Slope protection works	Turfing, Grating crib work, Gunite, Sprayed concrete, Gabion

4.6 *Recent developments - cut slopes*

During heavy rainfalls it is necessary to control the traffic of cars or trains. Rain gauges are effective for judging the critical condition to shut down the traffic. As a result, many rain gauges have been installed and many methods to judge critical conditions have been proposed. In the proposed methods several factors are considered (see, for example, ATC3 1997) :

1. Total rainfall (for example, two weeks up to the previous day)
2. Power of rainfall
3. Rainfall in one hour
4 Effective rainfall
5. Tank model method

Figure 4.19 shows one diagram to judge critical conditions by continuous rainfall and rainfall in one hour. Boundaries between failed and safe cut slopes and embankments estimated based on case histories are shown by curves. The limit to shut down the Japanese railways is shown by step lines.

4.7 *Examples of failures- embankments*

4.7.1 *Slope failures and slumps of expressway embankments during the 2004 Niigataken-chuetsu earthquake in Japan (partially quoted from Yasuda et al. 2008)*

On October 23 in 2004, the Niigataken-chuetsu earthquake, of Magnitude 6.8, occurred and caused serious damage to many structures and slopes in Japan. Six expressways were closed due

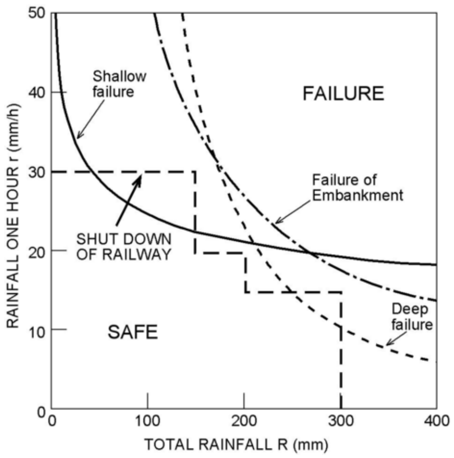


Figure 4.19: Diagram to judge critical conditions of failure of cut slopes and embankments for Japanese railways

to the earthquake. The total length of the closed expressways was 580 km. Emergency treatments were applied to the damaged expressway embankments by filling, placing and spreading. Then all expressways were able to opened for emergency vehicles about 19 hours after the earthquake because no serious damage was induced in expressway bridges and tunnels. About 13 days after the earthquake all expressways were opened for all vehicles.

Among the affected six expressways, the following two zones were severely damaged.

1. Between Muikamachi IC and Nagaoka IC of Kan-etsu Expressway (57.6 km) as shown in Figure 4.20, and
2. Between Kashiwazaki IC and Sanjyo-Tsubame IC of Hokuriku Expressway (50.3 km)

Most serious damage occurred in the following sections:

a. Kan-etsu Expressway: between Horinouchi IC and Echigokawaguchi IC (8.8 km), and between Yamamotoyama Tunnel and Yamaya PA (5.5 km) ,
b. Hokuriku Expressway : between Ohzumi PA and Ngaoka JCT (6.0 km)

The section between Horinouchi IC and Echigokawaguchi IC of Kan-etsu Expressway was constructed on the gentle slopes of hills. In contrast, the section between Yamamotoyama Tunnel and Yamaya PA was constructed on flat ground. In the former section, embankments were constructed mainly by "half-bank and half-cut" methods on the slope of the hills. Sliding of the filled embankments occurred at several sites during the earthquake. Where embankments were constructed by filling soils on level grounds in the latter section, large settlements of the embankments occurred.

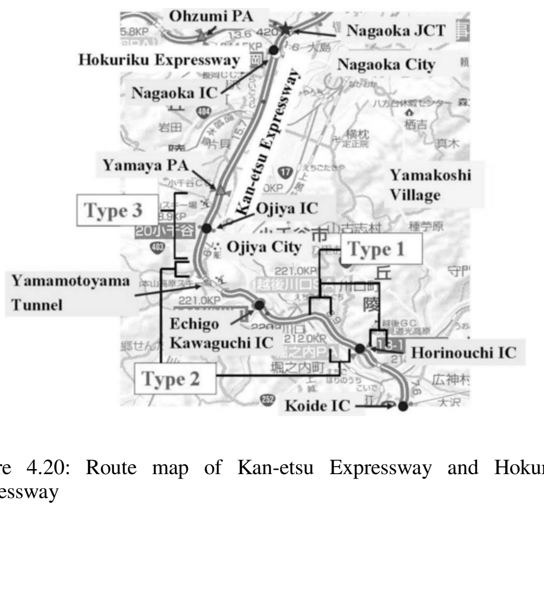


Figure 4.20: Route map of Kan-etsu Expressway and Hokuriku Expressway

Several seismic records were obtained in these severely damaged zones. The recorded maximum surface accelerations were 0.500g at Horinouchi Town, 1.757g at Kawaguchi Station, 1.532g at k-net Ojiya site and 1.029g at Ojiya Castle. Therefore it can be said that seismic motion in the severely damaged zones was very strong as the maximum surface acceleration was about 0.5g to 1.7g.

A section between Koide IC and Ojiya IC of Kan-etsu Expressway was selected to study the influence of type of embankments upon the damage. In this section, embankments were constructed by three methods: filling on a level ground, widening, and half-bank and half-cut. Percentages of the lengths constructed by these methods are 57 %, 5 % and 7 %, respectively. The other 31 % are cuttings, tunnels and bridges. Total lengths of damaged and undamaged embankments constructed by the three methods are compared in Figure 4.21. Lengths of damaged embankments seem to be two to three times the length of intact embankments regardless of construction method, and many sites were damaged even where the ground is flat.

In Japan, damage to road embankments is classified in three levels as shown in Figure 4.22, and this was applied to the Kan-etsu expressway. Serious damage occurred at half-bank and half-cut sections only. In the embankments on level ground, medium or minor damage dominated. According to the mechanism of failure, the damage of the Kan-etsu expressway embankments in the section between Koide IC and Ojiya IC, can be classified to three types as follows:

Type 1: Serious slide of the embankment on the sloping ground as schematically shown in Figure 4.23(a)

Type 2: Settlement of the embankment on the level ground without the deformation of the ground as schematically shown in Figure 4.22(b)

Type3: Settlement of the embankment and the culvert on the level ground with the deformation of the ground as schematically shown in Figure 4.223c)

Locations where these types of failures occurred are shown on Figure 4.20.

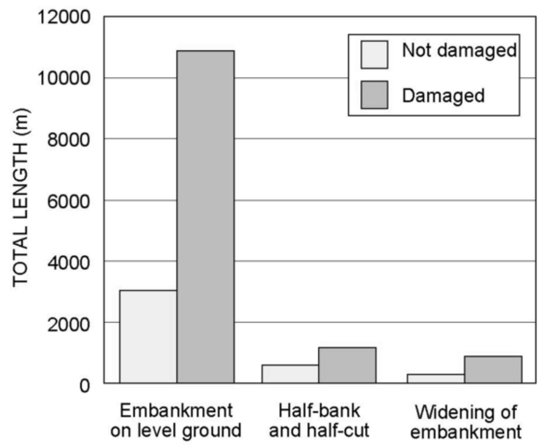
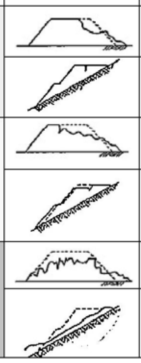


Figure 4.21: Total length of damaged and not damaged embankments constructed by three methods

In Japan, the current seismic design method for road, railway and river embankments is the seismic coefficient method based on circular slip surface analysis. However, it has become necessary to introduce performance-based design, in which it is necessary to evaluate the performance by the deformation of embankments, such as settlement. However, analytical methods to evaluate the deformation during earthquakes have not been fully developed. So, based on these case histories for road, railway and river embankments, new design methods have been discussed by several technical committees organized in the Japanese Geotechnical Society or other associations. In a few years, new seismic design methods will be introduced in design codes for road, railway and river embankments. In the new design codes, the following analytical methods will be introduced:

Type 1 – Embankment on sloping ground: Newmark's method.
Type 2 – Embankment on level ground: Dynamic response analysis or static residual deformation analysis.



Level of damage	Schematic diagram	Definition of damage
Minor		Surface slide of embankment at the top of slope only
		Minor cracks on the surface of a road
Medium		Deep slide of embankment or slump involving traffic lines
		Medium cracks on the surface of a road and/or settlement of embankment
Serious		Serious slump of embankment
		Serious slide of embankment

Figure 4.22: Classification of damage to road embankments

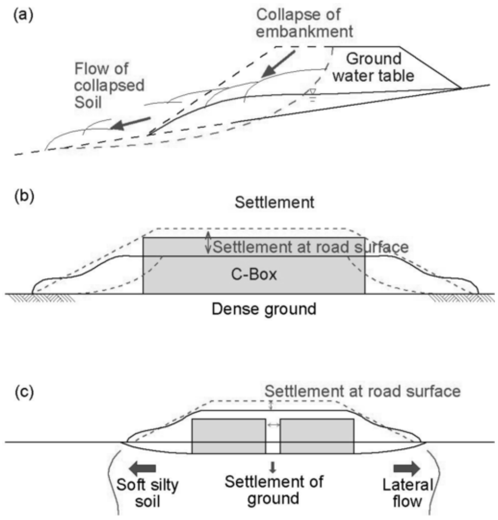


Figure 4.23: Classification of the damage to the embankment of Kan-etsu Expressway according to the mechanism of failure.
Type 1: Serious slide of the embankment on the sloping ground;
Type 2: Settlement of the embankment on the level ground without the deformation of the ground;
Type3: Settlement of the embankment and the culvert on the level ground with the deformation of the ground.

4.7.2 *Slope failures of a reclaimed area for housing lots*

1. Liquefaction-induced settlements of houses on filled grounds

An artificially filled housing lot at Kiyota district in Sapporo City was severely damaged during the 1968 Tokachi-oki earthquake. This might be the first experience of the damage to artificially filled housing lots due to earthquakes, in Japan. The maximum surface acceleration at Kiyota was estimated as about 0.08 g. The housing lot was constructed by cutting soils from hills and filling valleys as shown in Figure 4.24. There were 279 houses in the housing lot. Of them, 27.5% of houses were damaged during the earthquake. As shown in Figure 4.24 no house constructed on the cut ground was damaged. On the contrary, 56% of houses constructed on the filled ground settled and tilted. Ground water table in the fill was shallow and filled soil is volcanic ash sand. Therefore it is considered that the fill soil liquefied during the earthquake and caused settlement and tilting of houses.

In 2003, 35 years after the 1968 Tokachi-oki earthquake, a new Tokachi-oki earthquake occurred and caused damage to houses in this district again. Figure 4.25 compares damaged houses during the two earthquakes. In both cases, damaged houses were located on fill ground. However, houses damaged during the 1968 earthquake survived during the 2003 earthquake. After the 1968 earthquake underground conduits were constructed in the damaged zone, probably lowering the ground water level. This may be one of the reasons why the damaged houses survived during the 2003 earthquake.

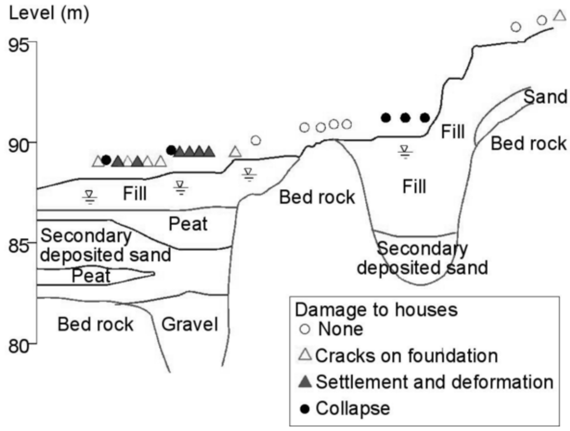


Figure 4.24: Soil cross section and damaged houses during the 1968 Tokachi-oki earthquake

At Utsukushigaoka district near Kiyota district, several houses settled as shown in Figure 4.26 during the 2003 Tokachi-oki earthquake. It can be judged that liquefaction occurred in this site because traces of sand volcanoes were observed around the damaged houses. The boiled soil was volcanic sandy silt. The damaged ground had been constructed recently by filling a channel in a hill zone. Therefore, it is considered that the filled soil liquefied and caused the settlement of the houses. Yasuda et al. (2004) measured the angle of inclination of the damaged houses roughly, as indicated on Figure 4.27. As shown in this figure, about seven houses settled and tilted more than 1°. According to the experience of the damage during the 2000 Tottoriken-seibu earthquake, inhabitants felt giddy and nauseous, and after the earthquake they could not live in the houses which tilted more than 1/100, as indicated in Figure 4.26

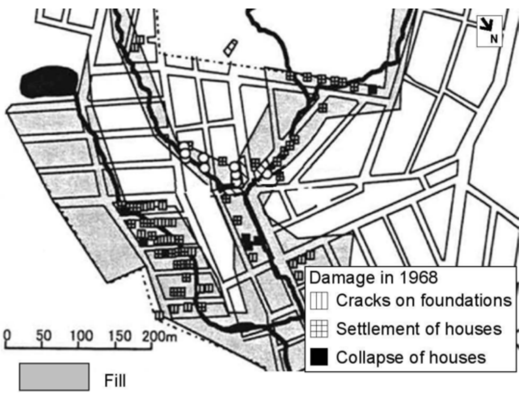


Figure 4.25: Comparison of damaged houses during two earthquakes

Figure 4.26: Settled houses due to liquefaction during the 2003 Tokachi-oki earthquake at Utsukushigaoka district

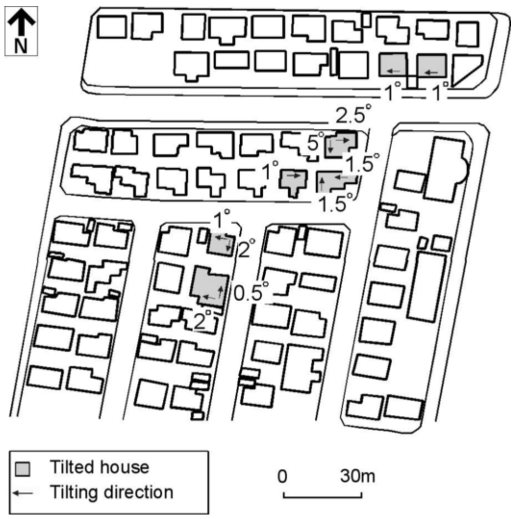


Figure 4.27: Rough measurement of inclination of settled houses

2. Collapse of houses due to failure of fill slopes

Several housing lots were damaged during the 1978 Miyagiken-oki earthquake in Sendai City in Japan, some of them affected by a severe slope failure of an embankment at Midorigaoka Ichi-chome housing lot. As shown in Figure 4.28, many houses collapsed due to the slope failure. In 1957 to 1958, the embankment was constructed by filling a valley. During the reclamation work, trees on surrounding hills were felled and thrown down into the valley. Rocks cut from the hill also were thrown into the valley, then soil cut from the hills was placed as schematically shown in Figure 4.29. Figure 4.230 shows a soil cross-section along the slope. The thickness of the fill was 5 to 15 m. The density of the fill was very low with 0 to 10 of SPT *N*-values. It is estimated that buried trees became humus within 30 years. Moreover, as big voids existed between buried rocks, groundwater flowed in the big voids and caused weathering of rocks. Then the bottom of the fill became very loose and caused the slope failure during the Miyagiken-oki earthquake.

During restoration, steel pipe piles, 318.5 mm in diameter, were installed to increase resistance against sliding, as shown in Figure 4.30. The steel pipes was strengthened by filling with steel H sections and concrete. The steel pipe piles were installed in two rows at 2m centres. A concrete retaining wall and drainage wells were also constructed. The safety factor against sliding during anticipated earthquakes was thereby increased to 1.2.

Figure 4.28: Collapsed houses due to slope failure at Midorigaoka in Sendai City during the 1978 Miyagiken-oki earthquake

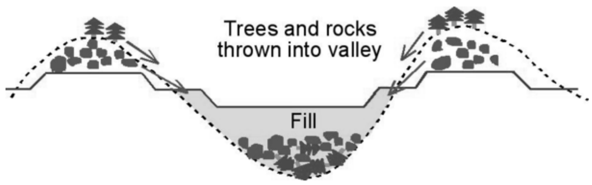


Figure 4.29: Procedure of reclamation work conducted at Midorigaoka

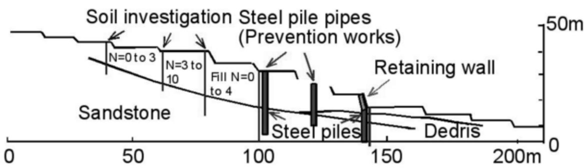


Figure 4.30: Soil cross section along failed slope (Tohoku Branch, JSCE)

Figure 4.31: Failed slope and damaged houses at Midorigakoka in Kushiro City during the 1993 Kushiro-oki earthquake

At Kayanuma district in Shibecha Town, land for housing lots had been developed by cutting the north slopes and filling in the south zones as shown in Figure 4.34. Four valleys were filled with a maximum thickness of 10 m. No special drainage conduits were put on the bottom of the valleys. Filled soils slid and nine houses were severely damaged during the 1993 Kushiro-oki earthquake as shown in Figure 4.35. Soil investigations by Swedish Weight Sounding and other tests were carried out after the earthquake to study the mechanism of damage. The fill soil was volcanic silty sand. Figure 4.36 shows a soil cross section along lines A-A' and B-B' which cross heavily damaged zones. The filled soil was loose with SPT *N*-

vales of around 10, and the lower part of the fill was saturated. It was difficult to recognize the occurrence of liquefaction during the site survey conducted just a few days after the earthquake, because the land was covered with snow and sand volcanoes could not be observed. However, detailed seismic and liquefaction analyses suggested the occurrence of liquefaction of filled sand. In the slope stability analysis, the safety factor against sliding was less than 1.0 if the seismic acceleration exceeds 0.2g Unfortunately the actual acceleration is not known because no seismic records were obtained around here. Deformation analysis by Tara 3 showed large deformation of about 1m, as shown in Figure 4.37. Deformation of the area could be estimated by FEM as shown in this figure because the slide was not large.

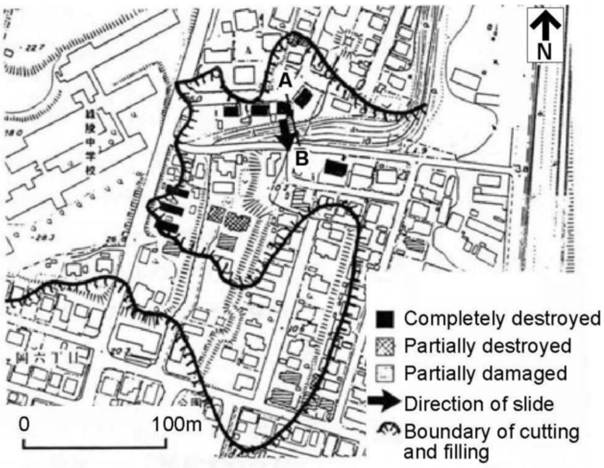


Figure 4.32: Location of damaged houses at Midorigaoka (JSSMFE,1994)

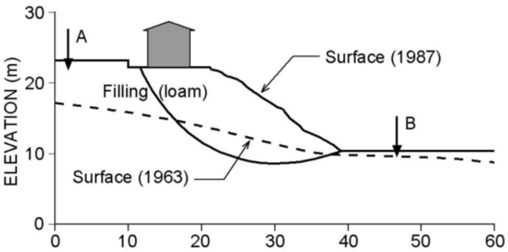


Figure 4.33: Mechanism of slope failure (JSSMFE, 1994)

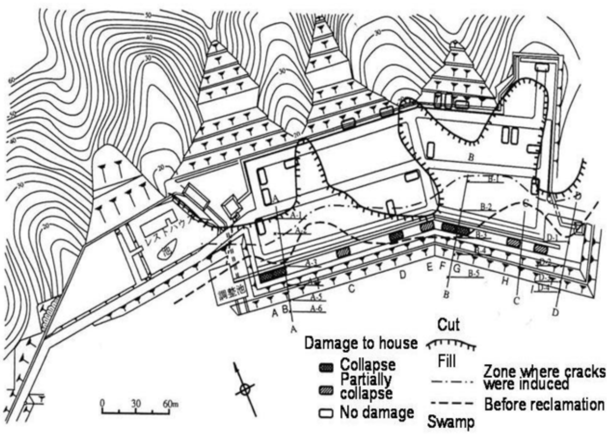


Figure 4.34: Filled zones and damaged houses at Kayanuma (JSSMFE, 1994)

Figure 4.35: Deformed bank and damaged houses at Kayanuma during the 1993 Kushiro-oki earthquake

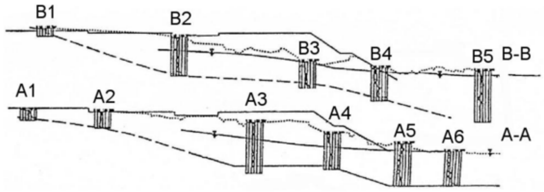


Figure 4.36: Soil cross section along the slope (JSSMFE, 1994)

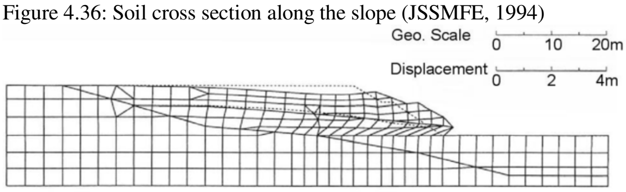


Figure 4.37: Deformation of bank analysed by Tara 3 (JSSMFE, 1994)

5 UNDERGROUND CONSTRUCTION

5.1 *Introduction*

This section of the paper addresses design and construction of underground structures, principally tunnels, and presents observations and findings based on experience gained over the past two decades during the design and construction of major railway, highway and water-transfer tunnel projects in the United States, Africa and Asia. During this time it has been possible to compare the performance of the as-constructed structures to the predicted performance as given by the design process. Excavation was typically carried out in widely varying ground conditions (weak-bedded rock, jointed hard rock, and soil-like poorly cemented rock with groundwater). It is therefore intended that the design guidelines could be applied, in general terms at least, to any tunnel excavated in any ground mass condition with any ground material type (hard rock, soft rock, bedded rock, soft ground, and soil) with or without ground water.

It may be queried as to why there is a basic review of tunnel design in a state-of-the-art paper. The purpose of this rudimentary review is to have designers and other design team members, inclusive of managers, realize that even with the advances of numerical methods and the tools available to assist in technical evaluations, it is still essential not to lose sight of the fundamentals. Common sense and a basic understanding of the fundamental physics involved, along with comprehensive checks during both design and construction are still imperative, even in the presence of all the computer wizardry.

The paper provides a basis for design of underground structures, specifically tunnels, by separately addressing the following: Basis of Design, Design Methodology, Design Codes and Analysis, Materials and Sustainability, Performance Monitoring and Construction Reviews and Future Developments.

Increasingly, concern for the environment and spiralling costs for commodities such as steel, concrete and fuel have also driven owners to consider and introduce specific requirements for sustainability. So one intention of this paper is to demonstrate how these sometimes conflicting requirements can be addressed through innovation and quality in the design, procurement and construction process.

5.2 *Basis for design*

5.2.1 *Definition of design*

The basics of tunnel design and achieving the functional requirements through the design of large span soft ground tunnels based on the sequential excavation and support method (SES) or Cut-Cover tunnels on a design-build or design-bid-build contract delivery project requires a well planned process that defines the objectives of the design as well as the process to follow while executing the design. This first essential requirement is usually addressed through the contract documents which govern the administration of any given project.

As more projects follow the trend of transferring project design and construction to a single entity and design service life increases to 80, 100 or 120 years, it becomes more important that contract documents clearly state the requirements expected of the design. Continuity in design process becomes paramount in ensuring that the finished product meets the expectations of the Owner. This is clearly evident for design-build contracts, but often the traditional design-bid-build projects also suffer from a lack of proper understanding of the development of tunnel design.

A tunnel design typically consists of six major attributes which define the required work:

- Each tunnel design is individual
- Each tunnel design requires the excavation method to be selected
- Each tunnel requires the decision to be either Drained or Undrained which defines the type of loading the lining must withstand
- Each tunnel design should define the standards and analytical tools to be used
- Each tunnel design typically has a unique set of “Owner” requirements
- Each tunnel design is not complete until the excavation is completed and the correct lining installed.

It may be asked, what is design? It may be thought of as any rational/verifiable/justifiable process whereby a concept is transformed into information that can be used to bring the concept to reality. In the case of a civil structure, design may be defined as that process, which is documented and can be audited, that determines information such as the shape (form) of the structure, the dimensions (size) of the structure, the thickness and strength of structural members that comprise the structure, etc. This information can be used to compute the time, money and resources required for construction of the structure. An optimum design may be thought of as one that not only maximizes the functionality and safety of the structure and minimizes its cost, both from an initial (capital) cost and a long-term (maintenance) point of view, but also takes advantage of known sustainability options available as well.

A very important requirement of a sound design is that there exists documentation of the process that was used, so that the design work can be examined later in relation to the measured or observed performance of the structure. Such an audit review would also be required for contractual, insurance purposes, etc, in the event of unsatisfactory performance of the structure.

During the design process typical Design Management Elements include but are not limited to:

- Collect data and develop concept
- Project hazard analysis
- Defining design requirements
- Define design protocol of codes and standards.
- Define quality processes
- Technical risk management
- Technical studies
- Interface management
- Technical review
- Environmental management
- Internal value engineering
- Producing designs and documents
- Clarifications
- Correspondence, mtgs. and communications
- Change identification and implementation

Careful management of these issues will assist in the development of an appropriately documented integrated design as is discussed later in this section.

5.2.2 *Types of underground structures*

For the purpose of this paper, a tunnel is regarded as an underground opening of limited span (width) and height (generally less than about 15 m), oriented in a horizontal/sub-horizontal direction, which is advanced linearly by rapid excavation techniques generally through a variety of geological materials. This section will not specifically cover other underground excavations such as shafts (which are oriented in a vertical/sub-vertical direction) and caverns, which have significant span and height and are more three-dimensional in nature.

Cut-and-cover construction, as the name implies, involves excavating a trench (cut), constructing the tunnel structure, and backfilling to restore the surface to its original condition (cover).

The concept of cut-and-cover construction dates back to at least 2180BC, when Babylonians constructed a brick arched tunnel, approximately 12 ft. high and 15 ft. wide in a trench across the Euphrates River as noted in the Questia Online Encyclopaedia (www.questia.com/library/encyclopedia/tunnel.jsp). The river flow was first diverted prior to construction and reinstated after the tunnel was backfilled.

With its inevitable surface disruption, cut-and-cover construction in highly trafficked urban environments can be intrusive. However, for a number of reasons – shallow alignment, large and/or varying cross section, prevailing ground conditions – which may make conventional tunnelling risky and/or uneconomic, cut-and-cover construction remains a viable and popular alternative.

Bored tunnels are typically employed when costs of cut-cover tunnels exceed that of mining techniques or surface disruptions are not tolerable. Tunnel bores may be advanced by a tunnel boring machine (TBM), generally with the use of a segmental concrete or iron lining, or by other techniques.

Approaches which do not use a TBM include SES – sequential excavation and support method, also known as SEM – sequential excavation method. This form of excavation requires providing support in the form of ribs and lagging or shotcrete and so is also known as sprayed concrete construction. In the highly stressed rocks of the European Alps, this approach has been developed as the NATM – New Austrian Tunnelling Method – in which support is provided to the opening based on the observed behaviour of the ground while adjusting to the revised stress fields.

5.2.3 *Performance requirements*

Owners’ performance requirements for underground structures have traditionally included safety, economy and durability for both construction and subsequent operation over and beyond the specified design life of the structure.

To assist with a common understanding of design terms, the following definitions are suggested. However any given project often has a set of predefined conditions to which the design must adhere. It is suggested that when this occurs the project team clearly define the terms and how they are intended to be used in the design of the structure. Following this suggestion will help to eliminate any misunderstanding between Owners, Designers and Contractors.

Tunnels must be safe as society demands security in the structures it inhabits or uses – thus tunnels are often classified into two categories:

- Inhabited space
- Uninhabited space

These definitions help clarify the level of risk that needs to be managed and where that risk may reside, i.e., life safety or project capital costs.

Having defined the intended use of the underground space, it is imperative to understand the requirements for which the structure is required to perform. The definitions of the types of analyses to be incorporated in the design provides a sound basis for guiding the design process. The definitions below serve well to guide the designer in this effort and are closely related to the functional requirements that are discussed below.

ULS – Ultimate Limit State Design may be viewed such that exceeding the limit results in failure. It is irreversible and may result in risk to loss of life and/or structure

SLS – Serviceability Limit State Design may be viewed as a hindrance or undesirable performance, but this may be reversible and does not risk loss of life or structure.

5.2.4 *Functionality requirements*

The type of tunnel to be designed and constructed often defines the requirements. Generally there are five types of tunnels and each category generally has its own specific requirements for functionality. The most common types of civil works tunnels are:

- Road Tunnels
- Railway Tunnels (Mass Transit, Commuter, Freight, High Speed)
- Hydraulic Tunnels (Water, Wastewater, Storm Water, Head/Tail Race)
- Utility Tunnels
- Pedestrian Tunnels

Each of the above tunnels generally requires a clear statement of performance requirements regarding but not limited to the following:

- Tunnel cross-section(s) (twin bores versus single bores)
- Design life
- Tunnel orientation
- Durability
- Alignment (horizontal and vertical)
- Lighting
- Portal locations
- Power
- Structure gauge/size
- Aerodynamics
- Water tightness /leakage
- Noise
- Ventilation/purging
- Vibration
- Drainage

Other issues which generally require early decisions include:

- Fife safety issues and requirements
- Security requirements
- Orientation of tunnel with regards to geological setting / design requirements
- Drained or undrained tunnel design (usually environmentally driven)
- Serviceability requirements (allowable displacement/deformation/cracking)
- Environmental issues / handling of tunnel spoil / groundwater pumping
- Project cost restraints / budgets / schedule

However, even with properly defined Owner Requirements there must always be a clear traceable path of evidence that shows that the client requirements for function, operation, finished size and shape have been achieved and that life-safety issues have been identified and duly addressed.

The Role of Geotechnics

An important difference between tunnel design and any other civil construction is the percentage that geotechnical parameters influence the design and construction and are quantifiable in comparison to the other design data. For example, in the design of a bridge or building, as illustrated in Figure 5.1, the geotechnical variables influence only the foundation, which is a minor part of the construction, and so give rise to little financial risk during construction. Whereas in tunnel design, these parameters influence the excavation and support which is the greater part of the design and construction; this results in a high risk on a major part of the financial cost associated with the work.

Hence the age old saying:

"Geotechnical Investigations?? ….. Pay Now or Pay Later".

Accordingly, the value of good geotechnical data and applied geotechnical engineering is essential to a successful tunnel project.

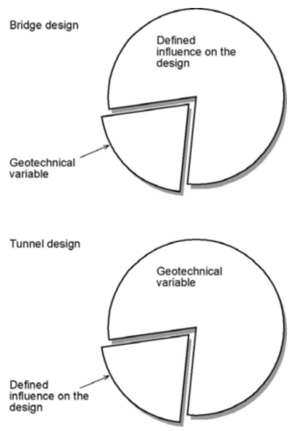


Figure 5.1 Role of geotechnical influence on design

5.2.5 *Site investigations*

A tunnel is a structure that is typically controlled geotechnically. The predominant loads acting on the structure are the ground pressure load and, if groundwater is present, the groundwater pressure. Other loads include dead load associated with the liner and any non-removable elements and the live loads as well as surcharge loads.

A tunnel design is only as good as the geological and geotechnical data. An important step in the design process is the determination of a "best estimate" of the anticipated geological and geotechnical conditions along the planned tunnel alignment. This information should include the following:

- Ground types (e.g. sandstone, silt, clay, sand, etc)
- Geological structure (bedding, jointing, faults, shear zones, etc)
- Ground mass strength and stiffness
- Groundwater characteristics (location of water table(s), permeability, etc).

An adequate site investigation program prior to construction should always be carried out for this purpose. The work must be sufficient to identify any adverse conditions which might occur along the tunnel alignment. Inadequate data often leads to excavation delays, slower progress, which increase the time required, and thus the cost, to excavate, support and line the tunnel.

It is often not possible to determine the exact distribution of conditions along the tunnel drive, so often "representative" conditions are established. These then serve as "benchmark" conditions that can be later compared to the actual conditions encountered. Historically, for any given region where geological conditions are equivalent and reasonably understood, it is possible to arrive at an estimated percentage distribution of ground types to be assumed for use in the "base" design for bidding. The actual design will require verification of ground conditions during construction with appropriate payment terms.

Once a prediction of representative geological and geotechnical conditions along the planned tunnel drive has been achieved, the next step is "geotechnical design". This is the process of predicting the response of the ground mass in each representative condition along the tunnel to the planned excavation of the tunnel. This prediction is required to assess what construction method, excavation method, support measures and ground treatments may be required to achieve stabilization of the excavation. For small span tunnels in "good" ground minimal, if any, measures are required and routine construction and excavation methods are possible. In "bad" ground, however, particularly where the tunnel is large in size, extensive complex procedures may be required.

5.2.6 *The role of water and water pressure*

Of particular note, the designation of a drained or undrained tunnel is not a definition of water tightness but it is a clear definition of the intended load which the tunnel shall be designed to resist. A clear understanding of this design requirement is essential to avoid catastrophic errors in the selection of the excavation opening size and the loading which the permanent liner shall be designed to resist. Undrained tunnels are typically designed to resist the full water pressure as defined by the designated groundwater table and geo-hydrological model. Whereas drained tunnels are often designed to resist only some portion (usually minimal/residual) of the pre-existing hydrostatic pressure.

The decision to build an undrained or drained tunnel should be based on actual requirements. Environmental issues should be the most significant factor contributing to the selection of an undrained tunnel. For example, it is understood that the German Rail preference has recently followed this policy of making this selection rather than the previous requirement for design and construction of only Undrained tunnels to reduce structure maintenance.

Often the Designers are permitted to propose design and construction of drained tunnels where they consider the design appropriate unless they are specifically prohibited by Contract. The following conditions typically need to be satisfied to allow for the design and construction of a drained tunnel:

- The reduction of the water table will not have adverse environmental effects (such as settlement of structures, significantly reduced stream flows, etc.).
- The inflows would not be so high as to cause maintenance problems.
- The ground water is not aggressive (<1.5 g/l of sulphates).

Drained tunnels are typically required to resist some amount of residual hydrostatic pressure. Often this pressure envelop consists of an allowance for a residual radial water head of 5m above the inner contour line of the inner lining. However, a proper understanding of the hydrogeological characteristics of the geology where the tunnel is driven is required. In some cases the residual radial pressure envelop approach is appropriate. In other cases a more conservative linearly increasing hydrostatic pressure loading may be more appropriate. The tunnel arch or vault area usually is not significantly affected by this amount of hydrostatic load regardless of the type of hydrostatic loading envelop applied. However, for non-circular tunnels, the tunnel invert will require significant reinforcement depending on the type of hydrostatic envelop applied and the geometry of the tunnel section.

Drained tunnels are typically required to have longitudinal groundwater drain pipes along each side of the tunnel with maintenance access not to exceed 50 m intervals. Where drained tunnels are constructed the drains typically consist of perforated pipe with either a gravel pack or no fines concrete for collection of the groundwater. The pipes are located near, but always below, the longitudinal construction joint at the interface between the crown lining and the invert 'corner'. The inner lining is tapered above and below to provide space for the drainage collection pipes.

Undrained tunnels are subjected to the full hydrostatic load as defined by the design groundwater level for the project or section thereof. Some projects establish more than one design groundwater level. In all cases save transient load cases, appropriate load factors should be applied to the groundwater pressures when designing the tunnel unless a "working stress" analysis is performed. Differing design procedures apply the load factors either to water pressure data in the analysis or to resulting stresses in the linings. Further debate of the merits of these alternatives will be valuable, involving both tunnelling and geotechnical disciplines. A very applicable paper by Bilfinger (2005) provides an approach to designing for groundwater loads on tunnel linings.

It is also relevant to note that tunnels do not have to be dry in order to achieve an undrained pressure loading. The only condition that is required to attain full hydrostatic load to the tunnel lining is a low relative permeability of the tunnel lining relative to that of the ground surrounding the tunnel.

5.2.7 *Waterproofing*

The Owner's requirements will usually dictate the required "dryness" of a tunnel. Water tightness usually has a direct impact on operation and maintenance costs for the Owner if the underground space is considered as inhabited. Accordingly, water tightness should be specified by an allowable leakage rate. It is rarely observed that a tunnel is truly dry. Hence there should be an allowance for acceptable leakage. This allowance requires an easy but reasonably discernable method to determine contract conformance.

During the course of a recent project it became clear that the contract documents were incomplete in addressing the requirements for water tightness for a rail tunnel where the contract merely stated that the tunnel must be "dry". Often complete dryness is not required to meet the functional requirements of the structure or that of the Owner. Accordingly the subject was duly reviewed and further clarified to provide a practical measure to achieve the Owner's functional requirement. To meet the intent of the project Design Specifications which stated that the tunnels shall be constructed to be completely "dry", it has been recommended that the use of the German Code DS 853 Class 3 as the objective criterion for evaluating compliance with this requirement. Unless very stringent watertightness criteria are required as for underground system control facilities, etc., this class of "watertightness" is considered adequate for most inhabited underground space as typically required for transit projects.

Typical water proofing systems consist of geofabrics, synthetic geomembranes with a series of water stops to compartmentalize segments of the tunnel structure and post grouting tubes to seal off leakage in the future. Designs typically include a waterproof membrane (typically 2.5 mm pvc) over the crown. A geotextile 'fleece' is usually provided outside the membrane and external waterstops cover the full perimeter of all transverse construction joints. Isolating waterstops are generally required at bulkhead locations to prevent water leakage from extending beyond the space between the bulkheads.

Where undrained tunnels are required or proposed, Designers typically provide a full perimeter waterproof membrane (typically 2.5 mm pvc) to create a fully "tanked" structure.

5.3 *Design methodology*

As with all project designs, a clear understanding of the design methodology must exist which is geared to addressing all the Owner's requirements and those issues not directly addressed by the Owner but which are the responsibility of the Designer to deliver a durable and safe system as defined by the Designer's profession.

5.3.1 *Integrated tunnel design process*

Tunnels must be economical firstly with regard to cost to design and construct and secondly with regard to maintenance. Most design decisions are, implicitly or explicitly, economic decisions. The life-cycle evaluation of the structure accounting for all costs and benefits arising from all the phases of the life of the tunnel must be taken into account. These phases include design, construction, operation and closure or transformation (salvage) of the facility. This concept is rapidly being advanced in the European countries under the designation DARTS (Rostam & Høj 2004) which is an acronym for "Durable and Reliable Tunnel Structures".

To account properly for the life cycle costs of the structure, an integrated design should account for all effects on the Owner and society over the complete lifetime of the structure as an integrated process. The level of detail requires adjustment to match the current phase of the planning and design and subsequent decisions, which affect the final structure and are required at various stages of the planning and design process. This approach requires that all parties involved with the project cooperate as a team and not as contractual opponents. An adversarial relationship during a project is a recipe for marginalizing the design, and risks missing the optimization of the project. The product, as seen by the Owner and Designer, will influence the design, specification, contracts, and the organization and structure between the various parties during execution and operation. Obviously this affects how the interfaces between the Owner, Designer, Contractor and other third parties are selected and administered over the life of the project to result in an integrated design. This is important as most tunnels are designed and constructed to serve the public and the complete life-cycle of the project should be optimized to their benefit

During the process of modelling and selecting design options and solutions, there is always uncertainty in the quantification each issue. All potential outcomes of the design process must be considered, evaluated and documented. Omission of any potential outcome may incorrectly favour solutions which do not provide real benefit.

Uncertainty may not be limited to the physical or statistical sense but also includes the relative belief in that uncertainty developing over the life of the structure. Examples of this sort of item include but are not limited to: 1) reliability of the structure performance with respect to limit state analyses, structural behaviour, durability, etc., 2) uncertainty of design loads developing and particularly in regard to load conditions or combinations, 3) uncertainty of costs associated with design modifications, geotechnical conditions, and environmental impacts, and 4) probability of undesirable events.

The design process may be viewed in the feasibility phase having decisions revolving around financial and environmental issues (politicians, bankers, government) whereas the detailed design and construction phases are focused on the technical engineering and constructability (future Owner/Operator and Contractor) aspects of the project.

Typically the project design is divided into five phases;

- Feasibility Study – choice whether to construct a tunnel or not
- Conceptual Design – choice of type of tunnel and alignment
- Preliminary Design – selection of initial design options, such as overall geometry and material grades
- Detailed Design – selection of final design options
- Construction Design – selection of construction options and as-builts.

For each level of the design process, the project begins with an idea and each subsequent phase of the design advances and requires more detail to be developed. With the completion of each phase of the project various elements of the design are established and design options assessed and presented which allow the "decision-maker" to determine the next course of action for the subsequent phases of the project. There are always elements of the design which must be worked out in detail in the earlier phases of the project, which then will be required to be properly assessed to evaluate the effects and resolve issues that will be encountered in later phases of the project.

5.3.2 *Design checks*

The design check list given below has attempted to show the hierarchy of the design steps which are required to complete the actual design of the structure once the Owner's requirements and functionality issues have been resolved.

- List functional requirements
- Select appropriate design codes of practice and design code protocols
- Determine design loading (empirical or numerical methods)
- Select applied design service loads / load factors / load combinations
- Perform structural analysis
- Establish required strength / internal forces / moments / stresses
- Select material strength and reduction factors based on strength requirements
- Size/re-size structural elements and determine design strength capacity
- Check design strength capacity against required strength demand
- Determine serviceability requirements
- Material stress (as applicable)
- Crack control
- Deflections/deformation
- Finalize design based on constructability and material availability
- Check, check and recheck the calculations and assumptions

Although the above may appear to be common sense, it is surprising how often errors in the detail of the design have resulted in large design, construction and contractual problems for a project. Peer reviews or group review sessions are always very helpful in identifying shortfalls in any design. Designers and various team members should learn that these reviews are geared to identify potential short falls or problems and determine the most appropriate resolution. The goal is to deliver the most optimal design for the project.

With today's computers, often the designers fail to grasp how the analytical tools currently available for use imply far greater precision than our ability to understand the ground properties for which the design must account.

Sensitivity analyses and an ability to understand the potential hazards and risks associated for the design can help identify where real savings in the design or real risks in the development of undesirable performance may exist. Often the sensitivity analyses will assist in identifying if additional design effort is really beneficial to the project or not.

In the past, inappropriate decisions in the application of codes and design methods have led to extraordinary events which may render a structure useless or far worse create a potential for injury or loss of life. A recent example is the collapse of the Nicoll Highway excavation in Singapore (Simpson et al 2008). This issue returns the reader to the beginning of this section where it is essential that the Owner, Designer and Contractor clearly understand the functional requirements of the structure and the performance limits that the structure demands.

Issues regarding proper modelling of construction sequences and load development, load combinations, material properties, serviceability requirements such as deflection/deformation and crack control as well as selection of appropriate ground and lining stiffness values are items which can often result in ambiguity in the definition of applied design loads and may dramatically affect the resulting performance of the tunnel structure.

There must always be a clear understanding of how the design has been undertaken and the construction sequencing must follow the design or the design must be re-evaluated to conform to the works in the field. All too often the designer has predicated the analysis and design on conditions which are unrelated to the actual works being performed in the field. Hence, the design is not finished until the tunnel has been constructed.

5.3.3 *The role of geometry*

Having the "User" defined requirements identified, there are always select issues which need to be accounted for in the design which will adequately allow sufficient space to construct the final tunnel structure following the excavation and support phase for SES/NATM type of tunnels.

There are several major design decisions which must be identified and resolved prior to the excavation phases such as:

- Pressure design requirements (if not environmentally required)
- Allowance for outer lining installation and deformations
- User requirements such as:
- Ventilation requirements
- Alignment constraints
- Water tightness criteria
- Emergency egress and niches/laybys
- Constructability requirements for excavation and lining installation

It is extremely important that there is a complete and full understanding of how these issues relate to the determination of the tunnel excavation opening size. Once a tunnel excavation is underway, it is very difficult and costly to have to increase the size of the existing opening as a result of any design errors or omissions. This issue has become more important as more underground works have been following the design-build project delivery system to accommodate new demanding project schedules.

5.3.4 *The role of maintainability*

Regular inspection and maintenance are required to ensure structures are performing as expected in achieving their anticipated design life, and to ensure any signs of distress or deterioration can be diagnosed and addressed in a timely manner.

The designer must consider how the cut and cover tunnel will be inspected and maintained over the lifetime of the structure. For a reinforced concrete box type permanent structure this is relatively straightforward. However, for cut and cover construction with integral support of excavation the finish of the diaphragm wall for highway tunnels in particular may not be desirable from an Owner's or user's perspective.

In such cases one solution has been to disguise the irregular diaphragm wall finish behind a secondary finish wall. Obviously the area behind the finish wall is difficult to inspect and maintain. However, the creation of sufficient space for man access between the diaphragm wall and finish wall purely for inspection purposes is expensive. Therefore the secondary wall is typically placed to leave an air gap of only a few inches between itself and the diaphragm wall. This gap allows sufficient space for a camera scope or similar remotely operated device to be inserted for inspection purposes.

Similar issues surround that of tunnel drain inspection and maintenance of drained tunnels. This inspection and maintenance requirement is critical to tunnel performance as the drains control the magnitude of load the tunnel structure has to withstand. Drain failure could result in significant over-load of the structure.

5.4 *Design codes and analysis*

Unfortunately, there is no known definitive Code identified for the design of underground openings. Generally designers use traditional building codes or other codes geared for above-ground structures and temper the approach and application of these codes based on underground design experience. Typical codes that are applied to underground structures are: ACI 318, ACI 224R-01 (2008), AASHTO (2002), BS8110 (1997), BS5400 (1988), Eurocodes (EN1992 2004; EN1997 2004) and other such related design documents.

In 2000, the British Tunnelling Society (BTS) and the Institution of Civil Engineers (ICE) issued the final draft of their "Specification for tunnelling" (published by Thomas Telford) This document refers to the following Eurocodes:

- Eurocode 1 (Basis of design and actions on structures, 1994/95)
- Eurocode 2 (Design of concrete structures, 1992)
- Eurocode 7 (Geotechnical design, 1996).

These codes, subsequently updated as EN1990, EN1992 and EN1997, have been used in a number of projects in the UK and throughout the world.

5.4.1 *Design code protocol*

It is essential to define accurately what codes will be applied, along with where and how these codes will be used to design the underground structure. Often misuse or alteration of the codes and design requirements result in an unfavourable outcome during the design process. Upon the commencement of each project, it is extremely important that the protocol for use of codes is clearly established and defined.

It is the desire of the authors that one day a proper code of practice will be developed for use in the design of underground structures and tunnels. Having an established code of this type will have a distinct advantage of resolving arguments between Designers, Owners and Contractors as the established code of practice will provide a sound baseline for the design process. This aspect becomes ever more important as useable above ground sites become unavailable, thus promoting the development and use of underground space.

More than other design works, there are often temporary structures that are required for the construction of the permanent structure. In the case of tunnels, this usually exists as braced excavations or the primary or outer-lining of tunnels constructed using the SES/SEM or NATM approach. The design protocols for these structures are no less important than that of the permanent or inner-lining. The only real exceptions to normal design requirements is in relation to the potential loading conditions likely to be encountered over the "useful" life of the structure and the requisite durability of the structure. Often requirements for displacement and deformation are not nearly as stringent as that for the permanent structure provided that the initial excavation geometry has allowed for these movements so as not to encroach into the required opening for the final structure or cause distress to third party utility and land owners. However, safety to the workers and to third parties must be maintained at all times and therefore the design of these temporary works should be subject to similar codes of practice to ensure that the structure(s) will perform satisfactorily.

5.4.2 *Design standards and codes*

The design standard selected for any given application can either be (a) a material code such as BS8110 or ACI318 which are principally building related standards, or (b) it can be related directly to the tunnel function – whether highway or railway tunnel – in which case in the USA the design would be in accordance with the American Association of State Highway and Transportation Officials (AASHTO) or the American Railway Engineering and Maintenance-of-Way Association (AREMA) standards respectively.

Table 5.1 indicates the differences in load factors for controlling loads for several design standards, each of which has been used in the design of cut and cover structures. By inspection, the use of one listed standard versus another can result in significant differences. For example, earth pressure factors, which constitute one of the largest loads applied to the structure can vary between 1.4 and 1.7, a difference of approximately 20%.

Note in particular:

a. AASHTO uniquely allows a factor of 0.65 to be applied for checking positive moments in frames.

b BS8110 Part 2 allows 1.2 to be used if applied to the "worst credible" earth and water pressures.

c The values for earth and, especially, water pressure are open to some interpretation.

Table 5.1. Ultimate limit state load factors

Design Code	Dead Load	Live Load	Lateral Earth Pressure	Hydrostatic Load
AASHTO	1.3	2.17	1.69[a]	1.3
ACI 318	1.4	1.7	1.7	1.4
AREMA	1.4	2.3	1.4	1.4
BS 5400	1.15	1.5	1.5	1.5
BS 8110	1.4	1.6	1.4[b]	1.4[b]
Eurocode 2	1.35	1.5	1.35[c]	1.35[c]

While the adoption of any of the above listed standards results in a functional design, in each case load combinations, load factors, and material strength reduction factors exhibit differences which translate into different structure sizes, different reinforcement quantities and hence different construction costs.

It is therefore of fundamental importance that the designer understands the cost implications for the adoption of any one standard versus another to identify the design standard which best reflects the function, durability and strength requirements of any particular project in the most economic fashion.

5.4.3 *Design analysis methods*

The majority of the analyses undertaken for design of cut and cover structures and mined tunnels are now computer based. Analysis models typically comprise two dimensional models which adequately represent the linear nature of the tunnels. Three dimensional analyses are typically not necessary but are useful for modelling specific locations where special conditions may exist such as rapid changes in cross section, large openings or appurtenances such as adits, alcoves, and equipment rooms.

A number of software packages are available using either structurally based software such as Structural Analysis and Design (STAAD) as manufactured by Research Engineers Inc which provide for two-dimensional plane frame analysis, or geotechnically based finite element/finite difference software such as Plaxis, manufactured by Plaxis BV or Fast Lagrangian Analysis of Continua (FLAC) as manufactured by Itasca.

Whereas the structural software is obviously focused on the structural elements of the design, the geotechnical software places greater emphasis on the simulation of the properties and behaviours of the surrounding soils and the interaction of the soils with the structure. Each of the software models and design approaches has its own benefits and limitations as outlined below.

For the structurally based software the following findings are typical:

- Models are simple and quick to develop and amend.
- It is relatively simple to model and simulate large numbers of individual loads and combinations of loads, using both service and ultimate load factors.
- Different factors can be applied to earth and hydrostatic loads based on the likelihood of a given design load to be exceeded.
- Models must also account for the variations in magnitude between vertical and horizontal loads as well as accounting for the loading to be favourable or unfavourable for structure performance.
- The effects of soil-structure interaction are modelled by springs which do not represent the ground well. This often results in the models overestimating the structure forces and hence the reinforcement requirements
- It is difficult to model adequately the complex sequence of construction associated with the installation of the support of excavation system. This typically requires the introduction of 'fake' forces to the models to reproduce the wall movements from the prior construction stage. This is particularly important in situations where the support of excavation is incorporated into the permanent structure.

With the geotechnical software the converse is true. For tunnel design, given the points identified above, FEM may provide a reasonable base to develop and understand the loads to be used in the design of the tunnel liner. On larger and more complex projects with multiple load types, a compromise is to use both modelling techniques. The structure software can be used to develop multiple load cases and combinations to identify which cause the most adverse conditions for stress. The controlling load combinations from the structural software can be replicated using the geotechnically based software, thereby promoting maximum design and construction economy.

5.4.4 *Examples of 3D analysis*

Yeow and Prust (2005) present an example of use of 3D finite element analyses to study the construction of a complex tunnel junction and its effect on existing tunnels. Figure 5.2 shows a detail of the junction, which was embedded in a mesh of 100,000 finite elements. The construction required about 90 stages of excavation. The ground movement was explicitly computed from the 3D tunnelling model without having to resort to the introduction of volume loss through stress relaxation or imposed volume change. Computed deformation were used for damage assessment. Ability to prepare data for analysis of this type and to carry it out in practicable timescales is still developing very rapidly. However, as discussed by Yeow and Prust, providing adequate models of the soil and structural materials remains a major challenge.

Further examples are presented by Simpson et al (2006). Figure 5.3 shows a situation at Stratford in East London, where the new Channel Tunnel rain Link tunnel was to pass beneath exiting metro tunnels of the Central Line. Since it is known that prediction of settlement due to tunnelling is extremely difficult, a special method was devised in which an empirically derived settlement field was imposed on a 3D finite element mesh which included the exiting tunnels. The details of the method are presented by Yeow et al (2005). Figure 5.4 shows the computed deformation of the existing tunnel which had a bolted steel lining, from which bolts were to be removed at various points. It was predicted that if the design ground loss occurred in the new tunnels the existing tunnel would not bend much but would shear at sections where the bolts were removed. In the event, actual ground loss and distortion of the existing tunnels were relatively small.

5.4.5 *Example – sprayed concrete lining in clayey silt*

The example shown in Figure 5.5 illustrates some of the problems to be considered in analysis of a sprayed concrete lining. The internal tunnel dimension is approximately 11m wide by 10m high, with a cover to diameter ratio (C/D) less than 2.0 It was excavated using an upper heading with temporary invert, with the bench excavated some distance behind the top heading. The primary lining of sprayed concrete is 350mm thick, and the secondary lining of cast-in-situ concrete is also 350mm thick.

Figures 5.6 and 5.7 show two forms of analysis which have been considered: finite elements and beam-spring models. The 2D finite element analysis uses a simple elastic-Mohr-Coulomb model and takes the mesh through all the stages of tunnel construction – heading and invert – with an appropriate allowance for ground loss assumed to occur during the excavation process. This inevitably leads to reduction in the final stresses on the tunnel lining as the soil arches over the de-stressed area of the tunnel, as illustrated in Figure 5.8. It is assumed that in the long term the primary lining degrades and is unable to carry bending stresses. If required, the formation of plastic hinges in the linings, both primary and secondary, can be modelled, though careful inspection of any plasticity which implies cracking in the secondary lining is needed by designers concerned about durability. Particular attention should be paid to the rotation capacity at the cracked joint.

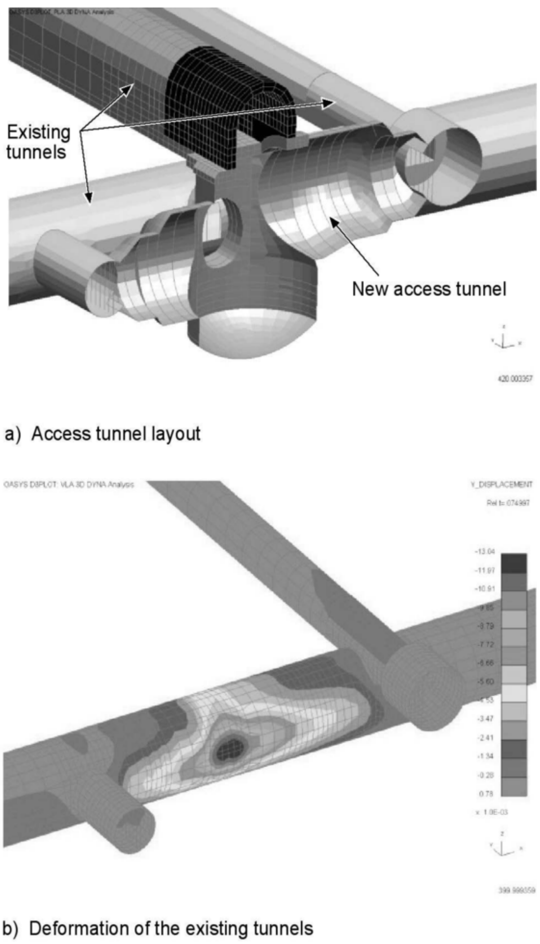


Figure 5.2: 3D modelling of SCL tunnelling operation (after Yeow and Prust 2005)

The spring model shown in Figure 5.7 can be used for an analysis following the general procedures recommended by Schulze and Duddeck (1964). This requires the designer to take decisions about the stress field within which the tunnel is placed, both in terms of the vertical and horizontal stresses, though these are modified as the lining flexes and the springs take up load. A particularly important decision is the choice of overburden pressure, that is, whether any allowance is made for the type of arching action suggested by the finite element analysis. Duddeck and Erdmann (1982) recommended that for shallow tunnels, C/D<3, a model without reduction of ground pressure at crown is appropriate. In general, for shallow tunnels (C/D < 2) the full overburden assumption is usually made. Such an approach leads makes much greater demands on the capacity of the tunnel lining than does reliance on the results of a finite element analysis of the type shown in Figure 5.8.

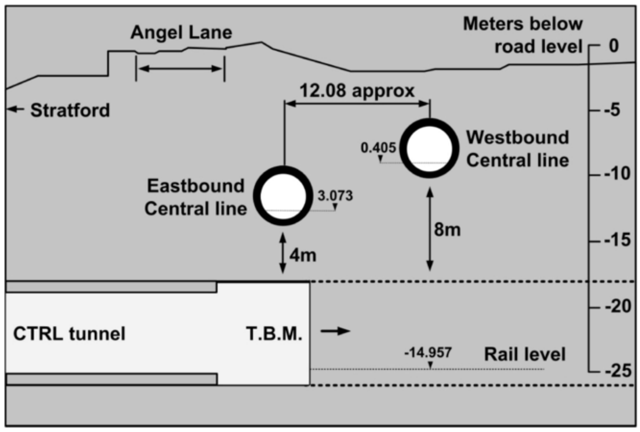


Figure 5.3: Intersection of the Channel Tunnel Rail Link and Central Line tunnels at Stratford, East London.

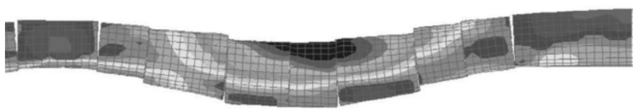


Figure 5.4: Computed deformation of a Central Line tunnel.

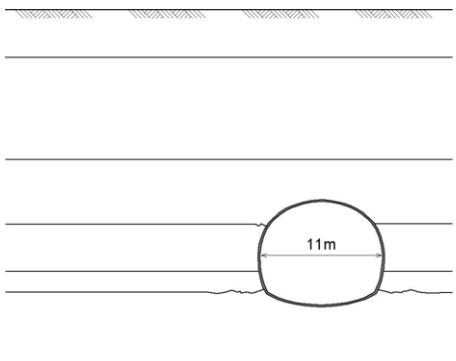


Figure 5.5: Tunnel construction in clayey silt

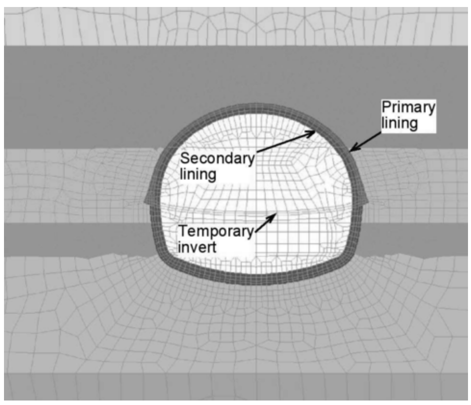


Figure 5.6: Detail of 2D mesh at the tunnel

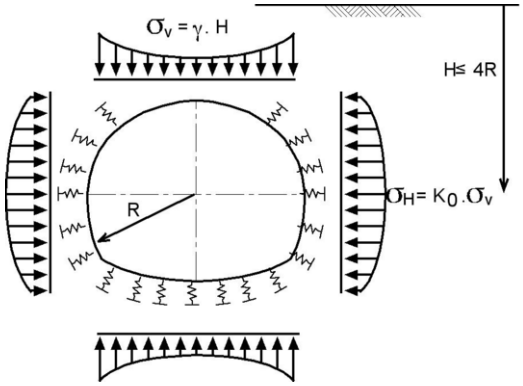


Figure 5.7: Beam and spring model

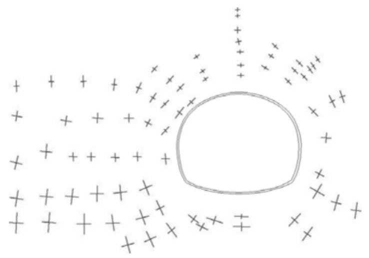


Figure 5.8: Principal stress plot showing arching

A further issue for both types of analysis is to decide how factors of safety, particularly load factors, should be applied. As noted in Table 5.1, the codes require that different factors are applied to dead and live loads. Some of them also differentiate between "unfavourable" and "favourable" dead loads, applying a factor of 1.0 to favourable dead loads. These distinctions are easily made if the factors are applied to the input data of the computations, but more difficult if an unfactored analysis is carried out, with the intention of factoring the resulting bending moments and thrusts.

Applying global factors to the forces derived from an unfactored analysis may significantly underestimate the adverse effects of live loading upon the structure. For instance the transient effects associated with heavy rain upon partially saturated soils may be underestimated if factors are simply applied to the loads derived from a calculation based on the soil properties in the partially saturated state. In that instance the adverse combination of both a reduction in strength and stiffness of the soil and a temporary increase in load may not be fully accounted for.

Fortunately, live loading is often less important to the design of tunnel linings, but the distinction between favourable and unfavourable dead loads can be controversial. Much of the loading on a tunnel lining is due to the difference between vertical and horizontal stresses in the ground. If one of these is deemed to be favourable whilst the other is unfavourable, the computed bending moments in the lining are increased considerably. More common practice is to regard the ground loading as coming from a "single source" (in the terms of EN1990), so that the same load factor is applied to all earth pressures. If this is the case, and if live loading is unimportant, the effect of applying load factors to input data of the analysis, or to the resulting bending moments, may be the same.

5.5 *Materials and sustainability*

In fairly recent years the use of cement replacement materials such as ground pulverised fuel ash (PFA) and granulated blast furnace slag (GGBFS) has become commonplace in the specification of reinforced concrete for cut and cover and other forms of tunnel. These materials are by-products from industrial practices from the power and steelmaking industries respectively.

The use of these replacement materials in reinforced concrete has been demonstrated to offer a number of advantages over Portland cement in terms of the durability and long term performance of the concrete. These advantages include:

- a denser concrete mix which improves watertightness
- reduced requirement for mix water
- increased resistance to chemical attack
- reduced heat of hydration during setting and curing

The final bullet point is particularly significant. Lowering the heat of hydration minimizes the temperature difference between ambient air temperature and the peak temperature within the concrete matrix during the setting process. By minimizing the temperature differential the incidence of problematic thermal cracking can also be minimized. This is of particular importance for massive concrete structures such as cut and cover structures which feature large concrete pours and onerous conditions of restraint at wall/slab interfaces as thermal cracks in theory will extend through the entire thickness of the concrete section.

However, there are some disadvantages to the use of cement replacement materials. The rate of gain of strength is slower than for Portland cement concrete, which results in forms being left in place for longer; material properties can vary necessitating that all supplies come from a uniform source where the properties are well understood. Analysis by Yazdchi et al (2005) suggested that the speed of construction of a small diameter tunnel with a sprayed concrete lining could be restricted by lower rate of gain of strength.

It can be seen from Table 5.2 that while there is extensive use of PFA in Hong Kong, Japan and the UK, in other major construction markets considerable room for growth in the use of cement replacement materials exists.

Table 5.2: PFA Production and Utilization 1995, after Meyer (2005) except UK data which is more recent, after Barnes and Sear (2004).

Country	Million Tons Produced	Million Tons Utilized	% Utilization
China	91.1	13.8	15.1
Denmark	1.3	0.4	30.8
Hong Kong	0.63	0.59	93.7
India	57.0	2.0	3.5
Japan	4.7	2.8	59.6
Russia	62.0	4.3	6.9
USA	60.0	8.1	13.5
UK	5.7	3.1	55

In terms of the long term performance of underground structures the use of cement replacement materials is recommended. In addition, as natural resources become scarcer and more expensive, the expanded use of cement replacement materials should be promoted.

The use of fibres, such as steel or polypropylene, in concrete for crack control and handling or to prevent explosive spalling of concrete under fire or an explosive event, respectively, is increasing throughout the world. The past performance in early projects has supported the industry-wide use of these fibres and as such, use of fibres is now given significant recognition and is widely employed on modern day projects.

5.5.1 *Instrumentation and communications*

While there has been widespread use of geotechnical instrumentation to monitor ground movements before and during construction, with the advent of the internet has "real time monitoring" – web based data provision and retrieval – many new systems are being developed everyday.

These systems offer a distinct advantage to construction of underground works in that immediate notification of adverse performance may be transmitted to the design/construction team 24 hours a day. This level of monitoring and notification should allow corrective actions to be implemented in a timely manner, thus averting undesirable events.

5.5.2 *Performance monitoring*

Tunnel excavation and lining installation should always include both outer lining and inner lining monitoring programs. For cut-and-cover structures this would correlate to support of excavation systems and the permanent structure, whereas monitoring devices are located for not only the temporary works but for the permanent works as well. By default, ground loading on the tunnel is directly related to the deformation of the ground and accurate and precise monitoring is one of the few tools available to verify the design assumptions regarding load and stability of the excavation.

The monitoring program can also include instrumentation to better understand ground movement beyond the limit of the excavation as well as loading on the outer and inner liner. Distinct instrumentation stations that are representative of sections of expected ground types (classes) are extremely helpful in verification of anticipated tunnel performance as well as for areas that have been determined to be difficult or problematic.

It is advantageous that the Designer specifically requires that these instrumentation stations are installed and that locations of installation are confirmed and agreed to by the Designer. Regular reading, interpretation and evaluation of the stations is also paramount. In mined tunnels, often the full ground load resulting from the excavation is not evident until the drive face is 4-6 diameters past any given location.

Predicted deformations of the outer lining as provided by analysis may be selected and used as “control” or “target” values during the excavation whereby these are compared to the measured deformations from the monitoring stations. Depending on the ratio of the calculated deformations to the measured ones, certain pre-agreed actions are implemented when the ratio reaches certain values. For example, “alert level 1” occurs if the ratio reaches 1/3, “alert level 2” if 2/3, and “alert level 3” if 1.0. The control values are determined for each stage of the excavation thus allowing for deformations to occur for each of the 3 standard stages of excavation, heading, bench and invert. A typical relationship between the “Control Criteria” and “Safety Control System” is provided in Table 5.3.

Table 5.3 Lining Performance Control Criteria

ALERT LEVEL	CONTROL VALUE	ACTION
Standard Operations	-	Standard Measurements
Level I Alert	1/3 Control Value	Increase Measurement Frequency, Conduct Site Inspections, Issue Strict Work Instructions
Level II Alert	2/3 Control Value	Strengthen Measurement System, Carry Out / Perform Minor Control Works (“Auxiliary Measures”)
Level III Alert	Control Value	Stop Excavation, Analyze Cause and Tendency of Deformation, Select Tunnel Reinforcement Measures

In view of the large excavation sizes, the tunnel cross-section is usually excavated in stages. Initially a top heading with an invert on or just above the tunnel spring line is excavated and stabilized, if necessary with a temporary invert of shotcrete to form a closed support ring. At a variable distance behind the face of about 40m to 150m, a bench is taken out below the top heading and supported. The final excavation of the floor, or invert, is taken out and supported, fairly close to the bench face, depending on the ground conditions. The closed support ring around the full tunnel section comprises the outer lining, which must stabilize the excavation before the inner lining is placed. The outer linings generally have not been considered as part of the long-term or permanent support of the tunnels.

During excavation, survey displacement monitoring stations should be installed close to the excavation face, typically at intervals of 10m to 20m, to measure the settlement and convergence of the outer lining and provide verification of the design of the outer lining. In poor ground conditions the spacing may be reduced to as little as 5m. Generally, each station consists of five targets, three in the top heading and two in the bench. The monitoring results should be used to verify the design assumptions for the outer and inner linings. An important aspect is that the information will be used to verify the assumed design ground loadings for the inner linings. Refer to Figure 5.9 for typical instrumentation layout.

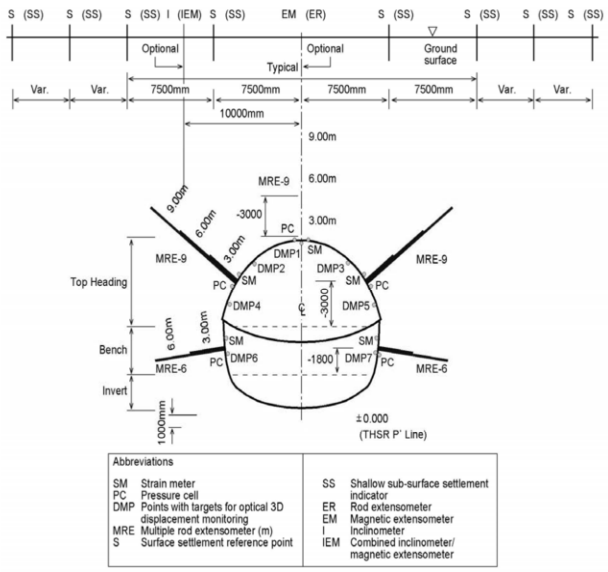


Figure 5.9 – Instrumentation Station

5.5.3 *Constructability reviews*

Constructability reviews are as essential to the design as are the drawings, specifications and supporting calculations. All the drawings and calculations will be for nothing if the design is not constructible. As an example, if difficult ground is encountered the tunnel invert excavations should be made deeper and more circular; this should be anticipated in cases where ground conditions are not well known or if it is known that poor ground conditions are likely to exist. This sort of flexibility in the design should be considered in the beginning and properly planned for, including layout of plant and equipment. The general concept of excavation equipment should be flexible to meet the changing demands of underground work during the tunnel construction.

Optimized reinforcement, especially in the arch lining, benefits the inner lining. High reinforcement content will increase the probability of honeycombing and voids. These reviews are often performed prior to tender award but they also continue through the construction phases as well.

The beginning of mined tunnelling should always be based on a conservative design. The observations during early site execution should accelerate the learning curve and optimize support requirements. As information is gathered, the Tunnel Construction Engineer and the Lead Geotechnical Engineer can work closely with the Designer who can then refine the analysis to support the decisions for lining design and support selection.

Precise Monitoring of deformation and interpretation of the deformation will help identify the best support solutions to minimize risk of collapse. However, deformations alone do not always tell the full story. Full instrumentation stations which are capable of identifying ground mobilization and lining stress allow for a full understanding of the interaction between ground strains and lining performance.

5.6 *Future developments*

In this emerging era of climate change and constrained resources it is incumbent on the engineering profession to design for value through economic use of materials and the promotion of engineered solutions which are durable and sustainable.

5.6.1 *Design standards and codes*

Development of a standard specific to below grade construction which is consistent with construction practices (or a number of standards dependent upon regions/jurisdictions) will help the review and acceptance of designs and construction performance.

As an example, appropriate load factors for groundwater levels are an important element of the design when hydrostatic pressures form a large part of the permanent load. The larger the percentage of total load resulting form water pressure should merit high scrutiny of the selection of load factor to be used with that load. Long term creep and concrete strain need to be reviewed in relation to the load applied..

Accordingly the UK National Annex to Eurocode 7 allows/encourages "direct assessment" of design values of ground water pressure (implying that no further factor is to be applied to these):

The partial factors specified in the National Annex to BS EN 1990:2002 [ie the normal partial factors on loads] might not be appropriate for self-weight of water, ground-water pressure and other actions dependent on the level of water, see 2.4.7.3.2(2). The design value of such actions may be directly assessed in accordance with 2.4.6.1(2)P and 2.4.6.1(6)P of BS EN 1997-1:2004. Alternatively, a safety margin may be applied to the characteristic water level, see 2.4.6.1(8) of BS EN 1997-1:2004.

An alternative approach is to calculate structural stresses for unfactored "characteristic" water pressures and apply factors to the structural stresses. There is considerable merit in checking both approaches in a design.

Accordingly design standardization and consistency of application of load factors/combinations etc. are very important aspects of design for underground structures and further development of the codes for design of such structures is warranted.

5.6.2 *Analysis*

More widespread integration of the temporary support of excavation (SOE) into the permanent structure helps to reduce environmental impacts associated with the work as it lessens the tunnel footprint and minimizes impacts upon utilities, and expensive real estate acquisition.

Soil-structure interaction models can be used to demonstrate the ability of SOE in the long term condition to relieve load on the cast in place structure, and correspondingly reduce structure size and cost. Use of integral SOE should be studied further to assist in the use of a hybrid solution for the permanent structure, while avoiding potential durability issues.

5.6.3 *Materials*

It is well documented that the concrete industry is one of the largest consumers of natural resources – relatively recent statistics suggest that the concrete industry consumes over 10 billion tons of sand and aggregates and 1 billion tons (1 trillion gallons) of water, not including water for wash down of mixers or curing on an annual basis (Transportation Research Board 2007, Mehta 2004, Meyers 2005).

In addition to the depletion of these natural resources, the production of cement in itself expends considerable amounts of fossil fuel and electrical energy. Annual production of cement is approximately 1.6 billion tons, and is expected to exceed 2 billion tons by 2010. The energy expended on the creation of one ton of cement generates an equivalent weight of carbon dioxide (CO_2), a principal contributor to global warming. The annual production of 1.6 billion tons (and rising) of CO_2 corresponds to approximately 7% of the global emission of this gas.

In addition, concrete construction debris from demolition constitutes a large percentage of solid waste disposal. In North America, Europe and Japan concrete and masonry rubble accounts for approximately two thirds of all construction and demolition waste. It is reported that in excess of 1 billion tons of construction and demolition waste is generated each year (Mehta 2001).

Clearly none of these practices is sustainable in the long term. However, solutions to mitigate or at least reduce their impacts are being identified and implemented. These solutions include the increased use of cement replacement materials – as identified earlier, these materials are significantly underutilized, and the use of recycled concrete aggregates should be promoted.

Typically concrete mixes are specified to contain roughly 30% of cement replacement material. With the benefits described previously proponents of the use of pfa/ggbfs in concrete production advocate that this percentage should be significantly higher and a content of 50-60% cement replacement should be sought. This increased volume of cement replacement materials will reduce requirements for cement production and correspondingly reduce CO_2 emissions.

An additional benefit of increased usage of cement replacement materials is the reduced requirement for mixing water. Concrete mixes using high percentages of cement replacement material have been demonstrated by testing to require 20% less mix water than corresponding mixes which are purely cement based. In many regions of the world water is a precious resource. A 20% reduction in concrete mix-water would equate to approximately 200 billion gallons of water saved per year. This is undoubtedly a significant volume (Mehta 2004).

5.6.4 *Contracting practices*

Due to the complexities involved with underground construction in an urban environment, it is time to move away from low bid construction which is prevalent over much of the world to a 'best value' approach whereby the contractors technical proposal forms an official part of the bid evaluation process (Pollalis 2006).

Consideration should be given to early contractor involvement in the design process, using a negotiated or target price contract approach as was successfully implemented on the UK Channel Tunnel Rail Link (Cathart 2005).

Under such a scenario qualified contractors would bid on design documents complete to a preliminary engineering level of detail. The selected contractor would subsequently work with the owner and designer to complete the design engineering. Through this process, the designer understands and can design to accommodate the contractor's proposed means and methods, the contractor provides constructability review of the design, and the owner receives a bid price with reduced contingency. Once the design is finalized, the owner and contractor renegotiate the contract price based upon the improved understanding and development of the project scope. In the event that an agreement cannot be reached, the parties can terminate negotiations, the contractor is paid for his design effort, and the contract can be rebid.

6 SEISMIC THEMES OF GEOTECHNICAL ENGINEERING INTEREST: BEYOND THE STATE OF PRACTICE

6.1 *Introduction: topics that have emerged from recent earthquakes*

This part of the report addresses issues related to what is called *"performance-based design"* of foundations against two earthquake–related hazards: (i) emergence of the rupture of a seismic fault underneath a structure, and (ii) strong dynamic shaking resulting from seismic waves emanating from the whole rupturing fault. Against the latter hazard (i.e. strong ground shaking) emphasis will be on the mobilisation of bearing capacity mechanisms for slender structures on shallow foundations. The topics chosen in this chapter have been prompted by observations in many recent earthquakes, most importantly from Northridge (1994), Kobe (1995), Kocaeli (1999), and Chi-Chi (1999). Valuable lessons of great significance have emerged from these events. Here are a few of them:

- Structures on conventionally stiff raft and box foundations *can* survive dip-slip and strike-slip fault ruptures producing offsets of the order of 1m beneath the structure. By contrast, pile foundations are directly affected by the rupture and, moreover, tend to impose on the super-structure the fault-induced differential displacements; both structural and pile damage are thus likely to occur.
- Even on very soft soil, a slender structure subjected to strong shaking may undergo severe rocking oscillations, accompanied by uplifting. Large deformations are thus induced in the soil and bearing capacity mechanisms are mobilised alternately under each half of the foundation. But the structure does not necessarily topple, and perhaps it does not even suffer detrimentally-large residual rotation and displacement.
- Geotechnical systems yielding only in one direction, such as gravity retaining walls and slopes, are quite sensitive to the nature and direction of seismic shaking and are thus greatly affected by phenomena such as *forward-directivity* and *fling-step*, often observed close to the seismogenic fault.
- Conventionally designed caisson quay-walls prove to be rather robust structures against toppling but they may experience (large) seaward displacement and rotation if the foundation soil is very deformable or, even worse, liquefiable; by contrast, the retained soil does not liquefy (contrary to a widely-held misconception) and will only exert moderate forces on the wall, of the same order as the pseudo-statically determined Mononobe-Okabe active forces.
- End-bearing pile foundations can perform very well in *"level"* ground, even if the surrounding soil were to liquefy massively, as long as there is no danger of buckling (usual case with the present-day large diameters of cast-in-place piles). However, in *"non-level"* ground, liquefaction triggers soil flow; as a consequence, piles could undergo severe "*kinematic"* lateral deformations and possibly bending failure. (*"Kinematic"* refers to ground-displacement induced stressing of the pile along its depth, as opposed to *"inertial"* loading which originates at the superstructure and is then transmitted on the top of the piles.) The possibility of buckling of small diameter end-bearing piles cannot be dismissed, however, as has been pointed out by Bhattacharya et al (2004, 2005) and Kerciku et al (2007).

Many theoretical and experimental studies have been initiated following these field observations, so that today the range of topics of interest in Seismic Geotechnics has greatly expanded. Two such topics are highlighted in this paper. The emphasis is on elucidating the analysis of seismic soil-foundation interaction in the presence of large soil deformations and near-failure conditions.

It is emphasized, however, that many of the ideas outlined in the following sections, although supported by some field evidence, are not yet to a sufficient extent developed to be directly applicable in design. Many more analytical studies and several large-scale or centrifuge experimental tests, as well as additional well-documented field evidence, are needed before these ideas are turned into reliable design methods accepted by the profession.

6.2 *Foundations interacting with a rupturing seismic fault*

6.2.1 *Problem and motivation*

"Strong" structures founded on the surface of, or at depth in, soil have often resisted successfully the loading induced by a rupturing seismic fault (Duncan & Lefebvre, 1973; Bray 1990). In the three Turkey and Taiwan earthquakes of 1999 numerous structures (buildings, bridge piers, retaining structures, electricity pylons, *dams*, tunnels) were located directly above the propagation path of the rupturing faults — strike slip, normal, reverse. Some of these structures performed remarkably well; others failed dramatically. These observations had a strong motivating influence to modify the pertinent clauses of seismic codes and to conduct further research. For it became immediately clear that the strict *prohibition of building in the immediate vicinity of active faults*, which the prevailing seismic codes have demanded, was unduly restrictive if not meaningless.

In addition to several geologic factors that contribute to such behaviour, the role of a soil deposit that happens to cover the rock appears to be significant. *If, where*, and *how large* will the dislocation emerge on the ground surface (i.e. the fault will outcrop) depends not only on the style and magnitude of the fault rupture, but also on the geometric and material characteristics of the overlying soil deposits. Field observations and analytical and experimental research findings (Bray et al, 1994a, b; Cole & Lade, 1984; Lade et al, 1984; Anastasopoulos et al 2007) show that deep and loose soil deposits may even mask a small-size fault rupture which occurs as a dislocation at their base; whereas, on the contrary, with a cohesive deposit especially of small thickness, a large offset in the base rock is likely to cause a distinct fault scarp of nearly the same displacement magnitude. One important finding of the above studies is that the rupture path in the soil is not a simple extension of the plane of the fault in the base rock: phenomena such as "diffraction" and "bifurcation" affect the direction of the rupture path, and make its outcropping location, the offset magnitude, and the shape of the deformed surface difficult to predict.

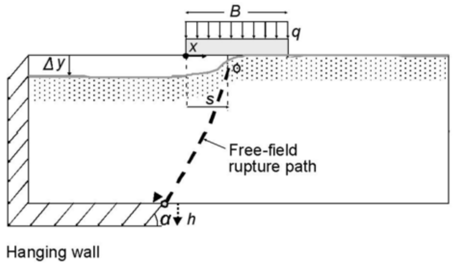


Figure 6.1: Configuration of the soil–foundation system subjected to a normal fault dislocation at the base rock.

Our main interest here is to study how a structure sitting on top of the fault breakout behaves. It turns out that an interplay occurs between the propagating fault rupture, the deforming foundation soil, the differentially displacing foundation, and the supported structure. Two different phenomena take place. First, the presence of the structure modifies the rupture path. Depending on the rigidity of the foundation and the weight of the structure, even complete diversion of the fault path before it outcrops may take place. Obviously, the damage to a given structure depends not only on its location with respect to the fault outcrop in the "free-field", but also on whether and by how much such a diversion may occur. Second, the loads transmitted from the foundation on to the soil tend to compress the "asperities" and smoothen the "anomalies" of the ground surface that are produced around the fault breakout in the free–field, i.e. when the structure is not present. Thus, depending on the relative rigidity (bending and axial) of the foundation with respect to the soil, as well as on how large the structural load is, the foundation and the structure experience differential displacements and rotation, different from those of the free-field ground surface. This phenomenon, given the name *Fault Rupture–Soil–Foundation–Structure Interaction (FR–SFSI)* is briefly elucidated below.

6.2.2 *Shallow foundations on top of rupturing fault*

The problem studied here is illustrated in Figure 6.1. A uniform soil deposit of thickness H at the base of which a normal fault, dipping at an angle α (measured from the horizontal), produces downward displacement (called "dislocation" or "offset") of vertical amplitude h. Note that the movement of the fault during an earthquake is in one direction only (and not oscillatory) and takes place rather slowly (on the order of tens of seconds rather than one-tenth of a second) – Ambraseys and Jackson (1984). The analysis is static and is conducted in two steps. First, fault rupture propagation through soil is analysed in the free field, ignoring the presence of the structure. Then, a strip foundation of width B carrying a particular superstructure is placed on top of the free–field fault outcrop at a specified distance S (measured from its corner), and the analysis of deformation of the soil–structure system due to the same base dislocation h is performed. The analyses are conducted under 2-D plane-strain conditions — evidently a simplification, in view of the finite dimensions of a real structure in the direction parallel to the fault. A limited number of 3-D analyses have shown that the 2-D approximation leads to slightly conservative results (i.e., it leads to somewhat larger rotation) (Gazetas et al 2007).

Among several alternatives that were explored, the FE model shown in Figure 6.2 produced results in excellent accord with several centrifugal experiments conducted at the University of Dundee for both steps of the analysis (Anastasopoulos et al 2007, 2009). A parametric investigation revealed the need for a long (B ≈ 4H) and very refined mesh (element size of 0.5 m – 1.0 m) along with a suitable slip-line tracing algorithm in the region of soil rupture and foundation loading. An elastoplastic constitutive model with the Mohr-Coulomb failure criterion and isotropic strain softening was adopted and encoded in the ABAQUS finite element environment. Similar models have been successfully employed in modelling the failure of embankments and cut slopes (Potts et al, 1990). Modelling strain softening was shown to be necessary; it was introduced by suitably reducing the mobilised friction angle φ_{mob} and the mobilised dilation angle ψ_{mob} with increasing plastic octahedral shear strain. With all the above features, the FE formulation is capable of predicting realistically the effect of large deformations with the creation and propagation of shear bands.

The foundation, modelled with linear elastic beam elements, is positioned on top of the soil model and connected to it through special contact elements. The latter are rigid in compression but tensionless, allowing detachment of the foundation from the bearing soil (i.e. gap formation beneath the foundation). The interface shear properties follow Coulomb's friction law, allowing for slippage. Both detachment and slippage are important phenomena for realistic foundation modelling.

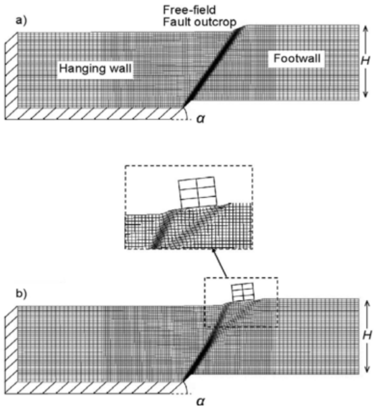


Figure 6.2: Finite element discretisation and the two steps of the analysis: (a) fault rupture propagation in the free-field, and (b) interplay between the outcropping fault rupture and the structure (termed Fault Rupture–Soil–Foundation–Structure Interaction, FR-SFSI).

A typical result showing the interplay between loose (ID = 45%) soil, rupture path, and a perfectly rigid foundation carrying a 3-storey structure is given in Figure 6.2. A base rock dislocation of 2 m (5% of the soil thickness) is statically imposed. The structure is placed symmetrically, straddling the free-field fault breakout (i.e. the foundation is placed with its middle coinciding with the location where the fault outcrops in the free field). A distinct rupture path (with high concentration of plastic shearing deformation and a resulting conspicuous surface scarp) is observed only in the freefield. The presence of the structure with its rigid foundation causes the rupture path to bifurcate at about the middle of the soil layer. The resulting two branches outcrop outside the left and near the right corner of the foundation, respectively. The soil deformations around these branches are diffuse, and the respective surface scarps are much milder than in the free field. Thanks to the substantial weight of the structure and the flexibility of the ground, the structure settles and rotates as a rigid body. The foundation does not experience any loss of contact with the ground; apparently, the foundation pressure is large enough to eliminate any likely asperities of the ground surface. As a result of such behaviour, the structure and its foundation do not experience any substantial distress, while their rotation and settlement could be acceptable.

Several tectonic, geometric, and material factors affect the interplay between an emerging fault rupture, the soil, and the foundation. For a fairly detailed parameter investigation relating to the subject, see Anastasopoulos et al (2007, 2009). This presentation will consider the consequences of this interplay for a rigid mat foundation, 20m wide, carrying a 2-story frame structure, and resting on top of a 40m thick deposit of dry sand. Two different densities are considered parametrically: ID = 45% (loose sand) and ID = 80% (dense sand). Typical values of (peak and critical-state) angles and dilatancy angle are assigned to each density, in addition to some other secondary modelling differences. For ID = 80%: φp = 45o , φcs = 30o , ψp = 18o, ψcs = 0o. For ID = 45%: φp = 32o , φcs = 30o , ψp = 3o, ψcs = 0o. Regarding their stiffness, both soil deposits are taken as "Gibson" type soils, i.e. with elastic Young's modulus increasing linearly with depth: E = 5z (MPa, m) for ID = 80% and E = 2z (MPa, m) for ID = 45%. The style and magnitude of the fault rupture play an important role in the response (and eventually the survival) of the structure. A normal faulting is considered here, emerging with an angle α = 45o on the surface of the base rock and with a dislocation (offset) having a vertical component h = 2m, that is 5% of the overlying soil thickness (H =40m).

Another important factor is the exact position of the foundation with respect to the outcropping location of the fault rupture on the ground surface. Since this location is affected by the foundation-structure itself, we use as reference the point O in Figure 6.1, where the fault outcrops at the free field ground surface. Having numerically determined point O, we specify as the position of the foundation the distance S of its left edge (i.e. the edge on the hanging wall) from O.

Two values of S are considered here: S = 4m and S = 16m. In the former case, the fault, if unaffected by the foundation, would have emerged near the left edge of the foundation; 80% of the foundation would have been located on the stable footwall. In the latter case, the fault would have emerged near the right edge of the foundation; only 20% of the foundation would have been located on the stable footwall. Figures 6.3 and 6.4 illustrate the response of the foundation in these two cases, for each of the two soil densities. (In both figures the vertical scale is substantially exaggerated; the slopes thus appear much steeper than in reality.) It is noted that in all four cases examined the foundation is lightly loaded (mean contact pressure po = 30kPa). The influence of po on the behaviour of the foundation-structure system will be explored later.

Several trends are worthy of note in the two figures:

1. For the first case, of the fault emerging near the left edge (*S* = 4m, or *S/B* = 0.20), this lightly loaded foundation causes only a minor diversion of the rupture path from its free-field position (Figure 6.3). This diversion is clearly noticeable only in loose soil (deviation to the left of about 1.5 m away from the footing centre, i.e. towards the hanging wall).
2. A profound consequence of the emergence of the fault under the foundation is the development of gaps between soil and foundation. On the dense soil nearly the whole left part of the foundation (≈ 4 m long) turns into a cantilever. The structural loads above this part induce a (substantial) bending moment — this indeed constitutes the major distress of the foundation from the rupturing fault. The survival of the structure depends on its ability to safely sustain this moment.
3. In dense soil, the observed asymmetric reduction in the contact area leads to an increase of the average normal contact pressure (to about 1.25 po) and to an unavoidable, if small, rotation of the foundation. All this culminates in a nearly triangular distribution of normal contact pressures with a peak of about 3po near the edge of the fault scarp, while the right edge of the foundation starts to uplift from the soil. This generates an additional cantilever at the right edge and further aggravates the rigid-body rotation of the foundation. This constitutes the second, "operational" distress of the foundation from the rupturing fault. Note, however, that for large enough values of po the local soil yielding will reduce the peak and lead to a more uniform pressure distribution.
4. On loose soil the size of the gap is restricted to 2.5 m, while along the remaining 17.5 m of the foundation width full contact is maintained. Hence, the bending moment of the cantilevered part is only 40% of the bending moment that develops under the same conditions on the dense soil. This favourable behaviour stems apparently from the greater compressibility of the loose soil and the ensuing depressing of the fault scarp, as is evident in the figure. However, an unfavourable consequence of the increased soil compressibility is the greater (by a factor of nearly 3) rotation of the rigid foundation.

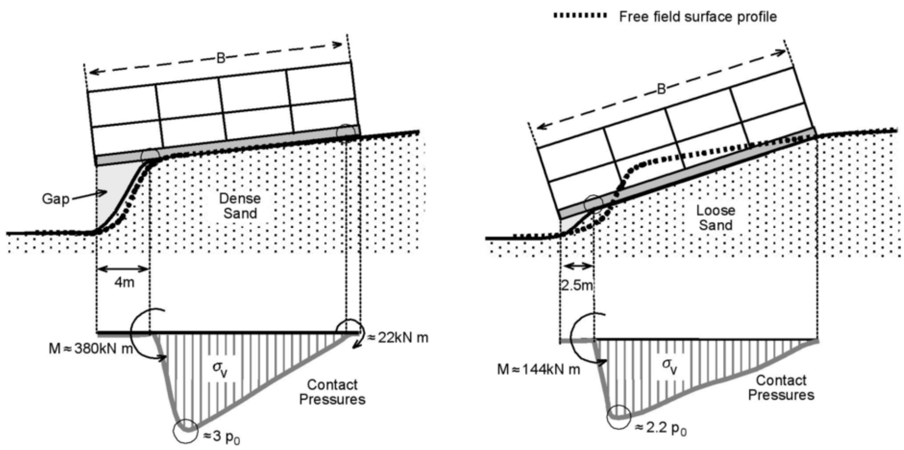


Figure 6.3: Response of soil surface and foundation due to a normal fault emerging at a distance S = 0.20B from the left edge under the foundation. (Vertical scale exaggerated.)

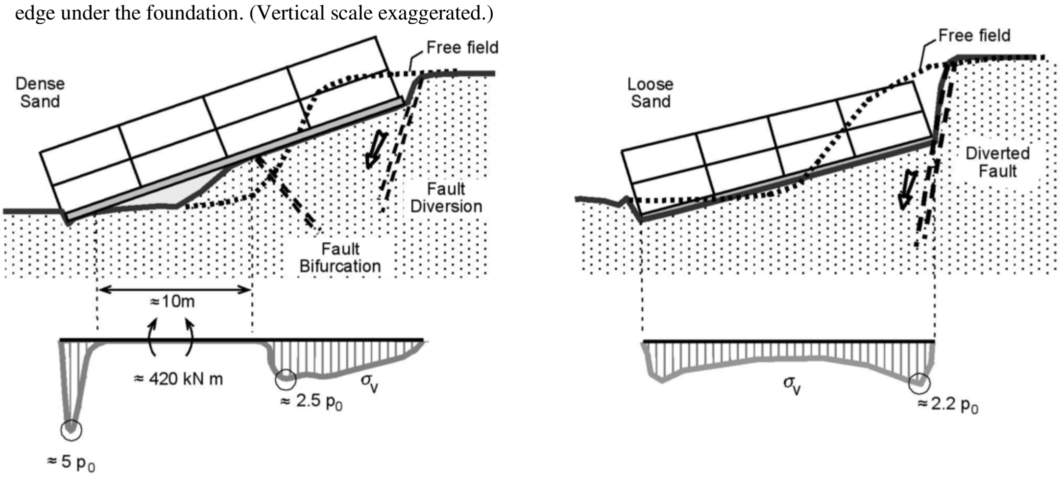


Figure 6.4: Response of soil surface and foundation due to a normal fault emerging at a distance S = 0.80B from the left edge under the foundation. (Vertical scale exaggerated.)

2. For the second case (Figure 6.4), of the fault emerging near the right edge (S = 16m or S/B =0.80), a substantial diversion of the fault takes place in both loose and dense sand: the fault deviates by 4 m towards the footwall in both cases, emerging beyond the foundation at the right-hand side. This significant theoretical finding has been verified with several observations of actual behaviour in the Kocaeli 1999 earthquake (Anastasopoulos & Gazetas 2007a,b). But the similarity in behaviour between loose and dense sand ends here. Significant differences are noted between the two cases:

- The fault scarp that is formed near the right edge of the building is far more conspicuous in loose sand.
- In dense sand the fault rupture undergoes bifurcation, with the secondary rupture branching to the left and emerging underneath the structure, not far from its centre.
- As a result, on dense sand, the middle part of the building loses contact with the bearing soil for about 10m, while the left and right part remain in contact for about 2m and 7m, respectively.

On loose sand, the response of the foundation is quite favourable: not only is the dislocation diverted and outcrops beyond the right edge as already noted, but full contact is maintained over the whole length of the soil–foundation interface. The distress of the foundation is thus significantly less on loose than on dense sand. Also smaller on loose sand is the (rigid-body) rotation of the foundation, thanks to the larger depressing of the fault scrap on the more compressible soil. Such a good response of a building founded on loose soil on the hanging wall is reminiscent of several success stories from the Kocaeli 1999 earthquake, especially of the building in Denizevler across the entrance from the Ford factory, near Gölcük (see Anastasopoulos et Gazetas 2007a). Figure 6.5 shows in some detail this building and the geometric characteristics of fault outcropping.

The above results are all for a specific small contact pressure, p_o = 30 kPa, at the foundation–soil interface, typical of a 2 story building. Although not shown here, the effect of an increase in the number of stories is quite beneficial on loose sand, and somewhat less on dense sand. The most significant benefits are the decrease of foundation rotation (and thereby of building tilting) and the elimination of a large part of uplifting. As a consequence, the survival of a heavy building founded on loose soil above a major fault rupture seems possible, in qualitative accord with numerous such success stories in several earthquakes.

The significance of the magnitude of the (average) contact pressure, p_o, is summarized in Figure 6.6. The figure gives parametric results for the size of the unsupported (detached) regions, *u,* and the effective (in-full-contact) regions, *b*, of the foundation. Both are normalized by the width *B* of the foundation. In addition to p_o, the examined variables include the location of the foundation with respect to the free-field fault outcrops, *S*, and the relative density of the soil, I_D.

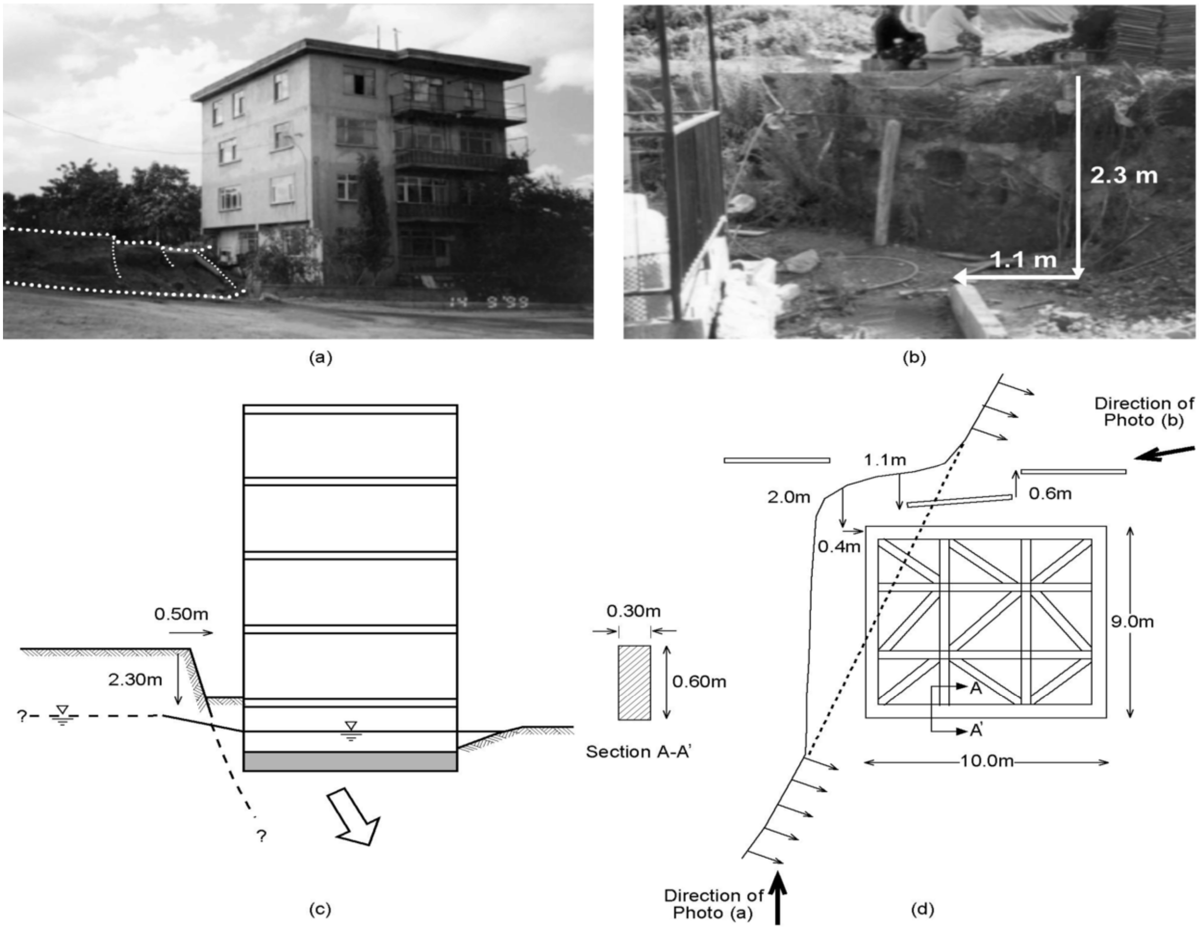


Figure 6.5: The remarkable performance of a 5-story building near Gölcük, Turkey, "on top" of the normal fault rupture in the Kocaeli 1999 Earthquake: (a) photograph showing the fault being directed towards the building; (b) photograph showing the vertical displacement reaching 2.3 m, along with a horizontal component of 1.1 m, measured on the torn-apart fence of the building; (c) simplified cross-section of the building and the fault; and (d) plan view of the foundation (box-type foundation with cross tie beams), along with the horizontal displacements measured around the building, and a sketch of the diverted fault. (after Anastasopoulos & Gazetas, 2007a).

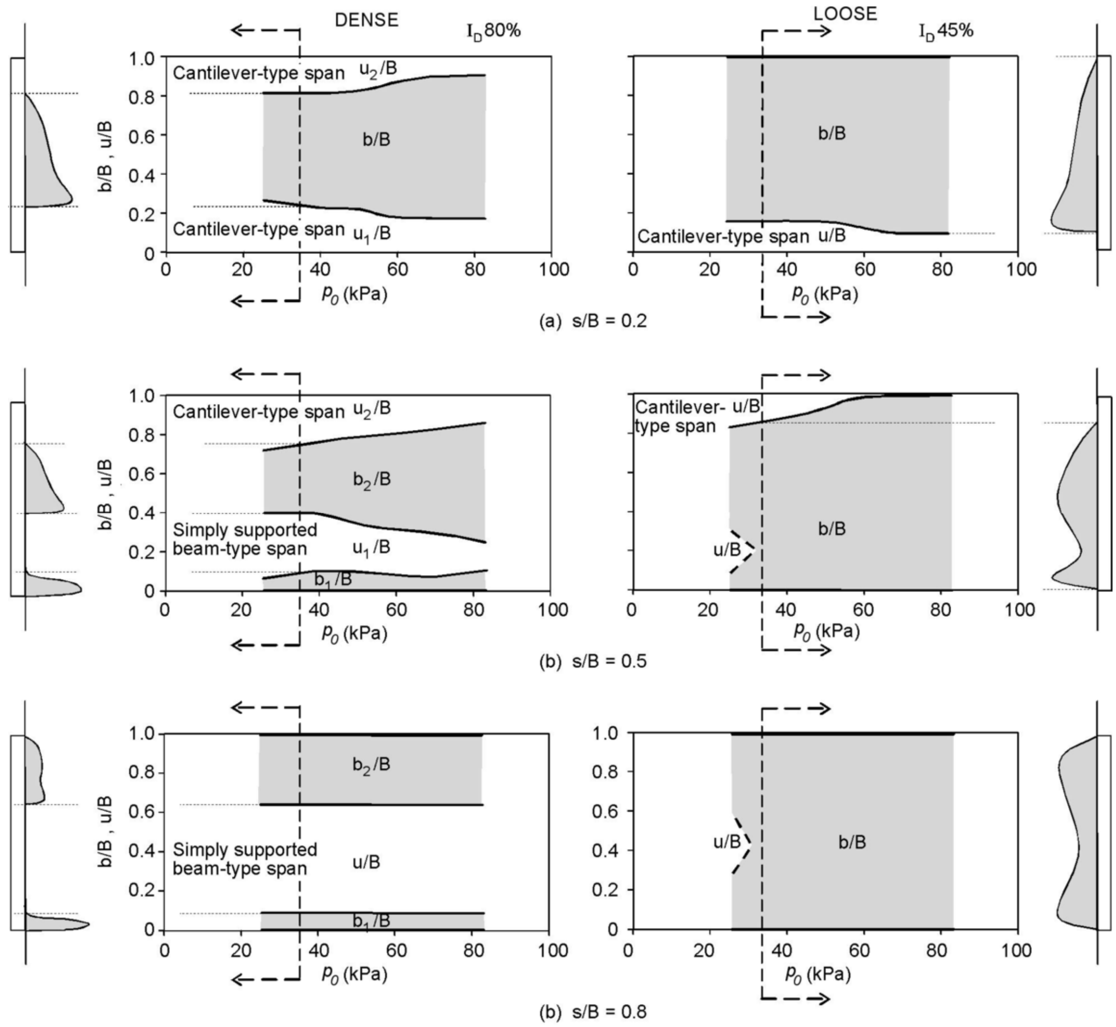


Figure 6.6: Summary of parametric results showing in normalised form which parts of the foundation remain in full contact (constituting the effective width, b, or b1 + b2 in certain cases) or separate from the soil (constituting the unsupported width, u, or u1 + u2 in certain cases). The fault ruptures at three different locations: (a) s = 0.2B, (b) s = 0.5B, and (c) s = 0.8B. The effect of surcharge load, po, is shown for the dense (left) and the loose (right) soil deposit. Indicative cross-sections of the foundation on dense and loose sand are shown on the left and on the right, respectively, showing the no-contact areas and crude sketches of the soil reactions.

The following trends are noted:

- On dense sand, for *S/B* = 0.2 the increase of p_o leads to an increase of the effective foundation width *b* from 0.60*B* to 0.75*B*, reducing the maximum (unsupported) cantilevered spans from u_1 = 0.25*B* to 0.20*B* (on the left side) and from u_2 = 0.2*B* to 0.10*B* (on the right side). For *S/B* = 0.5 there develop two unsupported spans: one (u_2) cantilevered on the right side (towards the hanging wall) and the other (u_1) doubly-supported under the left half of the foundation; thus there exist two areas of contact: a small (b_1) and a larger (b_2). The increase of p_o leads to increasing b_2 and decreasing u_1 and u_2 , while b_1 remains essentially the same. In stark contrast, in the case of *S/B* = 0.8 the increase of p_o does not seem to play a significant role.
- On loose sand, the effective width is invariably much larger than on dense sand. The most noteworthy effect of increasing p_o is in the case of *S/B* = 0.5: the unsupported spans disappear and full contact is established for p_o > 60kPa. But the biggest beneficial effect of increasing p_o is the reduction of the rotation experienced by the foundation.

The engineer could use Fig. 6.6 to preliminarily design the foundation raft. But this should be done only if he cannot reliably avoid building "*on the fault*". In view however of the unavoidable uncertainty on the exact location of the emergence of any fault, this means avoiding to build *"in close proximity"* to the fault

6.2.3 *Pile and Caisson Foundations*

The role of piles in supporting structures straddling seismic faults is far from clear. Circumstantial evidence from recent earthquakes has implicated the piles in some structural damage — see for example the analysis of the damage of the pile-supported Attaturk Stadium in Denizevler during the Kocaeli Earthquake (Anastasopoulos & Gazetas 2007). Systems "tied" to the different blocks of the fault may indeed be vulnerable. Reference is made to Gazetas et al (2007a,b) for a more detailed exposition of the behaviour of piled foundations under a normal fault rupture.

Rigid caisson foundations are clearly advantageous. The faulting-induced deformation causes a more-or-less rigid-body displacement and rotation that are in general smaller than those of surface or piled foundations. Reference is made to Anastasopoulos et al (2008a) for more information on the subject and an application to an actual bridge problem in Greece.

6.3 *Slender structures on shallow foundations: mobilisation of bearing capacity mechanisms*

6.3.1 *Conventional wisdom and the need for change*

Seismic design of structures recognises that highly inelastic material response is unavoidable under the strongest possible shaking. Displacements as large as 3 times the yield displacement (in earthquake terminology "ductility" of 3) or more are usually allowed to develop under seismic loading. This implies that the strength of a number of critical bearing elements is fully mobilised. In the prevailing structural terminology "plastic hinging" is allowed as long as the overall structural stability is maintained.

By contrast, a crucial goal of current practice in seismic "foundation" design, particularly as entrenched in the respective codes (e.g. EC8), is to avoid the mobilisation of "strength" in the foundation. In *structural* terminology: *no* "plastic hinging" is allowed in the foundation, below the ground surface.

Thus the conventional approach to seismic foundation design introduces factors of safety against sliding and against exceedance of ultimate capacity, in a way similar to the traditional static design. This approach involves two consecutive steps of structural and foundation analysis:

a. Dynamic analysis of the structure is first performed, in which the soil is modelled as an elastic medium represented by suitable translational and rotational springs (and, sometimes, with the associated dashpots). The dynamic forces and moments transmitted onto the foundation are derived from the results of such analyses, after the column forces have been reduced by dividing with a ductility-capacity dependent factor. (In EC8 this factor is called "behaviour" or "q" factor. It varies between 1 and 4; depending on mainly the ability of the superstructure to undergo safely large inelastic deformations.)
b. The foundations are then designed in such away that these transmitted horizontal forces and overturning moments, increased by "overstrength" factors, would not induce sliding or bearing capacity failure.

The use of *"overstrength"* factors is necessitated by the so-called *"capacity design"* principle, under which plastic hinging is allowed only in the structural elements — not in the below–ground (and hence uninspectable) foundation and soil. Therefore, structural yielding of the footing and mobilisation of bearing capacity mechanisms is not allowed. However, there is a growing awareness in the profession of the need to consider soil-foundation inelasticity, in analysis and perhaps even in design (see: Pecker (1998), Paolucci (1997), Martin & Lam (2000), Allotey & Naggar (2003)). This need has emerged from:

- The very large accelerations and velocities recorded in earthquakes, which would impose enormous inelastic demands to structures if soil–foundation "yielding" would not effectively limit the induced accelerations.
- Seismic retrofitting of a structure increases the shear capacity of some elements and hence the forces onto their foundations; it might not be feasible to undertake them elastically. A (stiff) concrete shear wall inserted to upgrade a frame carries most of the inertia-driven shear, and thereby transmits a disproportionately large horizontal force and overturning moment onto the foundation. If uplifting, sliding, and mobilisation of bearing capacity failure mechanisms are correctly taken into account, the shear wall "sheds" off some of the load onto the columns; the opposite is erroneously the case when such inelastic action is disallowed.
- Many slender historical monuments have apparently survived several strong seismic motions in their (often long) life. While under static conditions they would have easily toppled or otherwise failed, it appears that sliding at, and especially uplifting from, their base during oscillatory seismic motion has been a key to their survival. These phenomena cannot therefore be ignored.
- Compatibility with state of the art structural earthquake engineering is another reason to compute the complete inelastic lateral load–displacement or load–rotation response of the foundation system, to progressively increasing loads up to collapse. Otherwise the "performance-based" structural analysis, will be incomplete.

It is therefore logical to extend the inelastic analysis to the supporting foundation–soil system.

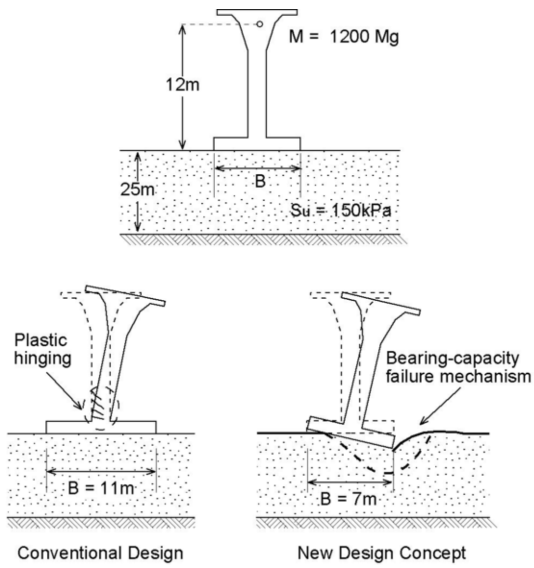


Figure 6.7: Top: problem geometry. Bottom: the two studied foundation solutions.

6.3.2 *Towards a new design philosophy: "plastic hinging" in soil-foundation systems*

Excluding structural yielding in the isolated footing or the foundation beam, three types of nonlinearity can take place and modify the overall structure–foundation response:

a. Sliding at the soil–foundation interface. This would happen whenever the transmitted horizontal force exceeds the frictional resistance. As pointed out by Newmark (1965), thanks to the oscillatory nature of earthquake shaking, only short periods of exceedance usually exist in each direction; hence, sliding is not associated with failure, but with permanent irreversible deformations, as will be shown in the subsequent section of this paper.
b. Separation and uplifting of the foundation from the soil. This happens when the overturning moment tends to produce net tensile stresses at the edges of the foundation. Thanks again to the oscillatory nature of the seismic shaking, the ensuing rocking oscillations in which uplifting takes place do not lead to overturning of the structure. There is not detriment to the vertical load carrying capacity and the consequences in terms of induced vertical settlements may be minor. Moreover, in many cases, footing uplifting is beneficial for the response of the superstructure, as it helps reduce the ductility demands on columns.
c. Mobilisation of bearing capacity failure mechanisms in the supporting soil. Such inelastic action is almost unavoidable with uplifting of the foundation. In static geotechnical analysis large factors of safety are introduced to ensure that bearing capacity modes of failure are not even approached. In conventional seismic analysis, bearing capacity is avoided thanks to an "overstrength" factor of about 1.40. (Note that this factor multiplies the (maximum) design

moment at the base of the superstructure; it is this increased moment that must be carried by the foundation–soil system.) The oscillatory nature of seismic shaking, however, allows again the mobilisation (for a short period of time) of the maximum soil resistance along a continuous ("failure") surface. No collapse or overturning failure occur, as the applied causative moment quickly reverses, and a similar bearing-capacity mechanism may develop under the other edge. The problem again reduces to computing the inelastic deformations, i.e. permanent rotation.

6.3.3 *Results of a comparative study*

To illustrate the interaction between soil, foundation, and structure under strong seismic shaking mobilising inelastic deformations in the soil we have selected the system portrayed in Figure 6.7. A single-column concrete bridge pier, 3m in diameter and about 12m high, carries a deck load of 12MN (mass of 1200Mg). It is founded with a surface square foundation (side B) on a 25m thick stiff clay deposit (S_u = 150kPa).

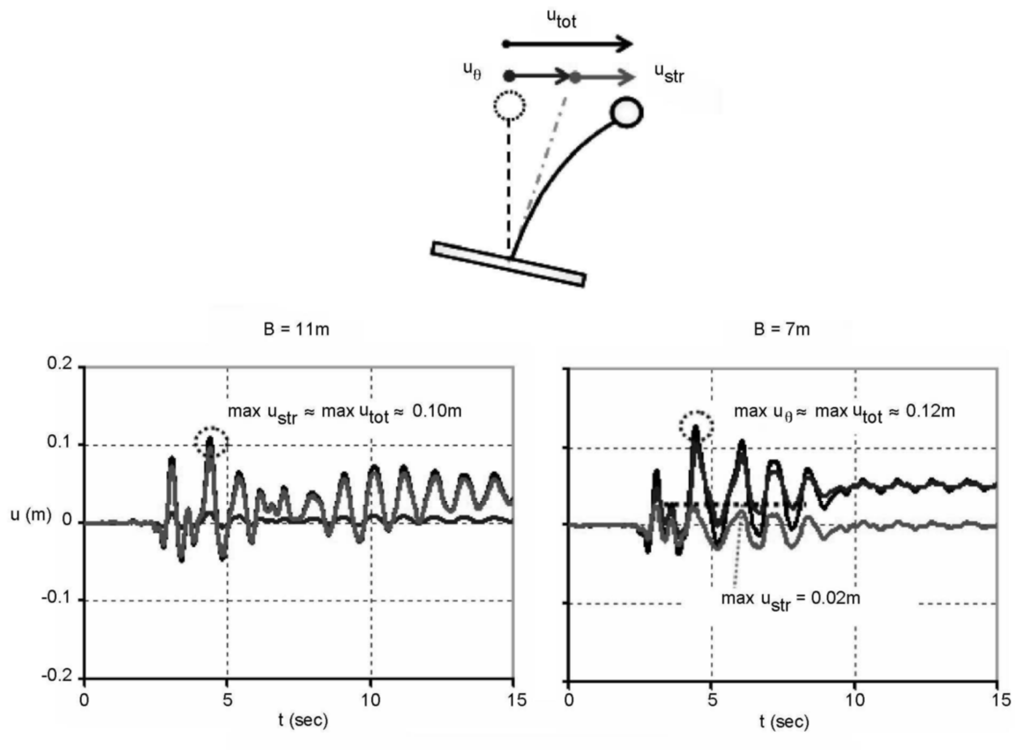


Figure 6.8: Computed displacement response time-histories for the two designs of Figure 6, for the "small" intensity (Kalamata 1986) motion.

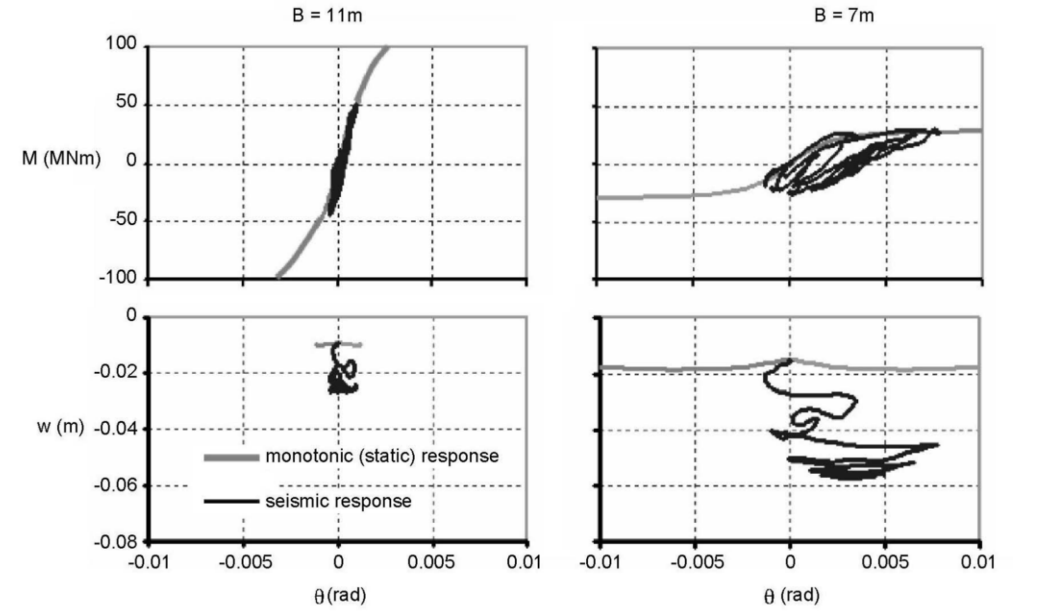


Figure 6.9: Computed moment–rotation and settlement–rotation hysteretic response of the two foundations of Figure 6.6, for the *"small" intensity* (Kalamata 1986) motion.

Two foundation solutions are examined:

a. B = 11m. This derives from a conventional slightly-conservative design, which leads to a large static bearing–capacity factor of safety (FSV ≈ 5.6) and a pseudo-static bearing-capacity factor FSE ≈ 2, for a code-specified design spectrum having A = 0.24g (horizontal), corresponding to soil category "B", and an estimated "behaviour" or "q" factor of 2. Although the value of 2 is high for a seismic FS, it was chosen in the anticipation of much-stronger acceleration histories than those specified by the A = 0.24g code spectrum. It is thus quite possible that during a design or stronger–than–design event structural (bending) plastic hinging will develop at the base of the column, with a rather minimal inelastic action in the soil or the soil–footing interface — a typically prudent conventional design.

b. B = 7 m. This corresponds to the new design concept, where significant plastic deformation is allowed to take place in the foundation–soil system, to the point of mobilisation of the bearing–capacity failure mechanisms. These may develop alternatingly on either side under the footing, as large cyclic overturning moments arise during shaking. This design is barely adequate under static vertical loads (FSV ≈ 2.8); under the design earthquake it leads to a pseudo-static FSE = 0.50 — well below unity to be acceptable within conventional engineering thinking. It is therefore expected that during shaking by a design–level, and especially by an above–design–level, ground motion the soil "failure" mechanisms will develop. The question is what the consequences will be for the foundation and the superstructure, and how the computed responses of the two systems of Fig. 6.6 differ from one-another.

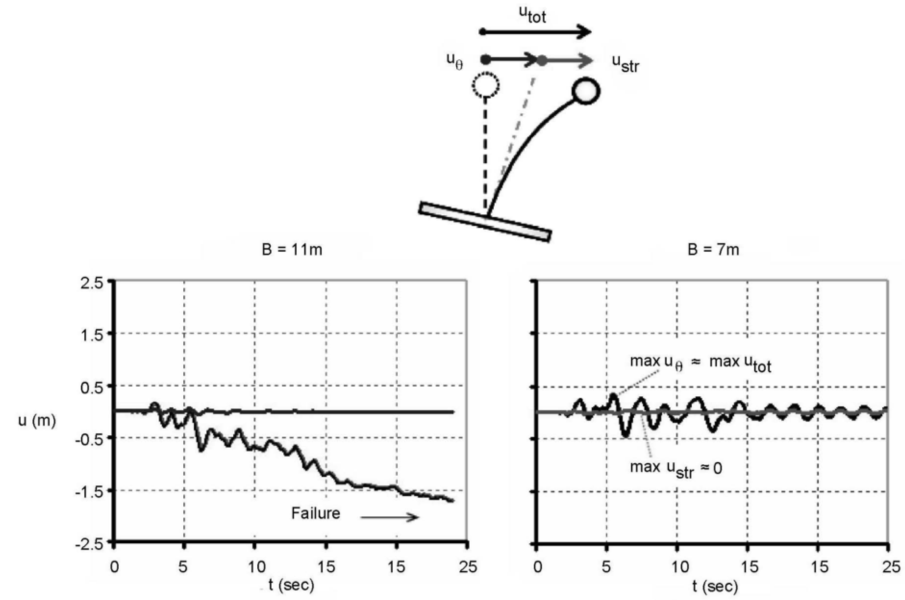


Figure 6.10: Computed displacement response time-histories for the two designs of Figure 6, for the *"high" intensity* (Takatori 1995) motion.

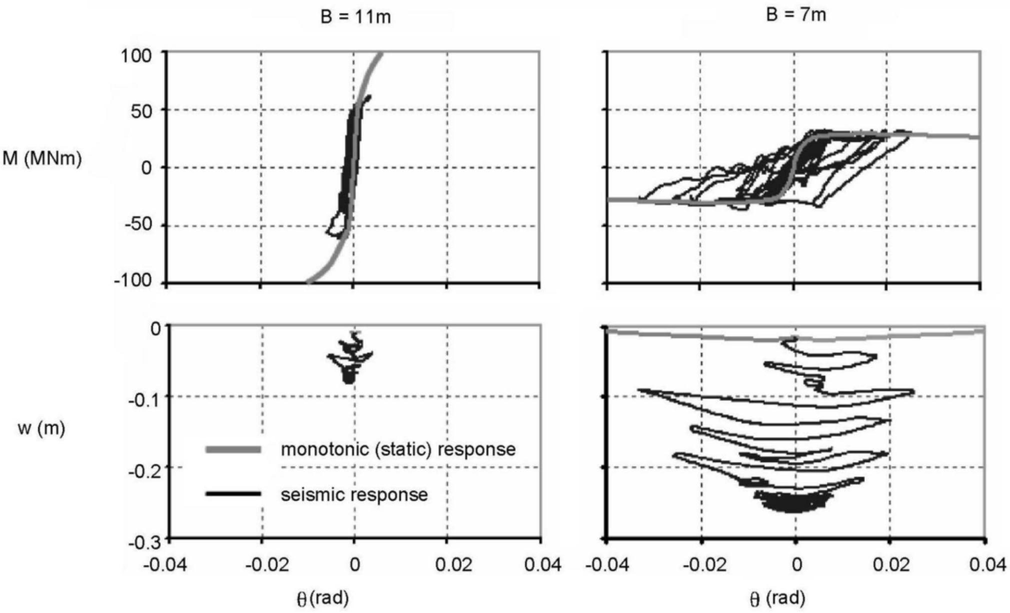


Figure 6.11: Computed moment–rotation and settlement–rotation hysteretic response of the two foundations of Figure 6.6, for the *"high" intensity* (Takatori 1995) motion.

This is demonstrated with Figures 6.8 - 6.11. Two different real accelerograms are used as excitation:

- The Kalamata Administration Building record of the 1986 Ms = 6.2 earthquake (Gazetas et al 1991). With an A ≈ PGA ≈ 0.26g but a response spectrum with values smaller than those of the design code spectrum at the period range of interest, this motion will be referred to as "Small" Intensity excitation.
- The Takatori record of the 1995 MJMA = 7.2 Kobe Earthquake, which in addition to its high PGA, 0.63 g, has spectral values in excess of 1.5g over a very wide period range (0.30 sec – 1.20 sec) . It thus undoubtedly constitutes a "High" Intensity excitation — substantially larger than a design excitation.

Figures 6.8 and 6.9 show the results for the *"Small" Intensity* shaking. Specifically, Figure 6.8 plots the time histories of the main components of superstructure displacement:

- the displacement of superstructure mass due to foundation rocking: $u\theta$ (in blue)
- the (additional) displacement of the superstructure, representing distortion of the column due to bending: ustr (grey)
- the algebraic sum of the above two: utot = ustr + $u\theta$ (black)

The lateral displacement of the foundation is not shown, as it is quite secondary for the particular chosen slender system.

Figure 6.9 refers to the response of the foundation. It plots the dynamic overturning moment-rotation, $M - \theta$, relation and the dynamic settlement–rotation, w–θ, relation for each of the two foundations. The conclusions emerging from the figures are clear:

- The conventional foundation (B = 11 m) experiences very small nearly-elastic rotation, with a small accumulation of cyclic settlement (≈ 2.5 cm). By contrast, the column experiences large distortion, almost 10 cm, as if the structure were responding on a fixed base.
- The "daring" foundation design, B = 7 m, experiences very large rotation, with $u\theta$ reaching 12 cm. The significant inelastic action in the soil is reflected in the highly-hysteretic $M - \theta$ relationship, as well as in an appreciable accumulated foundation settlement (≈ 6 cm). By refreshing contrast, the structural distortion has been limited to merely 2 cm — indicative of almost elastic column response.

In similar fashion, Figures 6.10–6.11 compare the results for the *"High" Intensity* (Takatori) excitation. The following is a summary of important conclusions from these plots:

- The conventional foundation design, B = 11 m, with too little help from the ("unyielding") foundation, cannot cope with the huge accelerations of the Takatori record. mechanisms. The maximum rotation θmax reaches 0.036 rad, corresponding to a substantial deck displacement $u\theta$ ≈ 50 cm. Accumulated settlement: wmax = wres ≈ 26 cm. The superstructure, however, hardly deforms and thus remains safe despite the much-larger-than-design ground shaking. With a warning: the above significant displacements (26 cm and 50 cm) may imply such large differential settlements between adjacent foundations and differential displacements between adjacent piers that indirect structural damage is unavoidable. Note also that the developing accelerations in the structure are also smaller when the soil is yielding and the foundation uplifts — an additional benefit effect from foundation plastic "hinging".

One might arguably consider the above footing–related deformations as excessive. Notice, however, that these were peak values; the residual rotation and displacement appear to be very small — for this particular excitation, undoubtedly coincidentally, they almost vanished!

In conclusion, inelastic "failure" mechanisms could be allowed to develop in soil-foundation systems designed for strong seismic excitation. Mobilisation of such mechanisms does not usually lead to failure; it may in fact prove quite a beneficial way to save the structure.

Two supporting case histories are mentioned here: (a) the settlement of slender buildings in Adapazari during the 1999 Kocaeli Earthquake, despite mobilisation of bearing-capacity "failure" mechanisms (as tentatively explained by Gazetas et al 2003). (b) the behaviour of the Kobe harbour breakwaters with no overturning and only small permanent lateral displacements and rotations; whereas by contrast, the uni-directionally moving soil-supporting quay walls (caissons identical to the breakwaters) suffered huge seaward displacements and rotations (Sekiguchi et al 1996, Dakoulas and Gazetas 2007).

7 CONCLUDING REMARKS

This paper has presented highlights from recent developments in some important areas of geotechnical analysis and design. Whilst analysis plays an important role, the process of design is wider, incorporating empirical methods based on carefully recorded experience, together with adequate factors or margins to provide society with the level of safety and serviceability recognised as needed on the basis of experience and tradition. Provision of adequate contractual processes are also an integral part of design.

In the development of codes and standards, attempts are being made to rationalise the safety processes, though reference to past practice is still deemed to be irreplaceable. Similarly, the valuable results of advanced analytical techniques must be reviewed and interpreted in the light of a thorough understanding of the real behaviour of structures interacting with the ground.

The authors hope that pooling experience from different continents, varying geologies, onshore and offshore will enable geotechnical engineers to provide more reliable analysis and to derive sound designs.

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