Prediction, monitoring and evaluation of performance of geotechnical structures

Prévision, contrôle et l'évaluation de la performance des structures géotechniques

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ABSTRACT

A review is presented on three core activities of geotechnical engineering: the prediction, the monitoring and the evaluation of performance of geotechnical structures. Four types of geotechnical structures are contemplated, each with its particular type of response: Foundations (mainly deep), Earth Fills, Supported Excavations and Tunnels. Each structure is treated separately, reviewing typical response, evaluating response prediction, reviewing procedures for response monitoring, evaluating performance and discussing specificities of application of Interactive Design, together with some selected case histories description. A final section is provided on geotechnical instrumentation, where requirements for planning and selection of instruments to be used are reviewed and recent fibre optic monitoring developments are discussed. A section with summary and conclusions is finally added.

RÉSUMÉ

Une révision des trois plus importantes activités de génie géotechnique -- la prévision, le contrôle et l'évaluation de performance des structures géotechniques -- est présentée ci-dessous. Quatre types de structures geotéchniques, chacune avec ses particularités, sont abordées: Fondations (profondes, en particulier), remblais, excavations soutenues et tunnels. Chaque structure est traitée séparément, avec révision: de sa réponse typique, de l'évaluation de la prévision de cette réponse, des procédés pour l'instrumentation de performance, et de l'évaluation de la performance, en discutant les spécificités d'application du Projet Interactif (Interactive Design), sous la lumière d'antécédents choisis. La section finale s'addresse à l'instrumentation géotechnique, en révisant les conditions pour la planification et le choix des instruments à employer, avec attention aux développements recents d'instuments à fibre optique. Le travail se ferme avec un résumé et les conclusions.

Keywords: Prediction, monitoring, performance evaluation, deep foundations, earth fills, supported excavations, tunnels, Interactive Design, case histories, geotechnical instrumentation, fibre optic instruments.

1 INTRODUCTION

The ability of predicting has always been highly praised by mankind. Those in the past that showed, to any extent, such ability, quickly top ranked in primitive societies. Oracles were consulted by knights and kings to decide courses of action. Common people looked for omens on striving for survival. Shaman could anticipate the future and change it by their influence on the good and evil spirits. Presently, science nurtures the compelling need of society to anticipate things, by its continued quest for understanding the causes of any process in nature. By knowing its causes, it is believed that the effects of a natural process can be forecast and, willingly, controlled for the sake of mankind.

As in any other profession, engineers are expected to provide anticipation of the performance of structures they design and build. Their successes in fulfilling this wish result largely from their ability to numerically model their prototypes, when the latter are built under their strict and stringent specifications. This is not the case for most geotechnical structures, in which a considerable and important part of its components cannot be specified, as they are defined by nature and require assessment and investigation. Society is understandably shocked when a geotechnical structure fails, as it perceives failure as a human error. Geotechnical engineers, on the other hand, when facing failure, tend to ponder how ground misbehaved. Both society and geotechnical engineers might be wrong. Peck (1981), pointed out that ninety percent of dam failures occurred not because of inadequacies in the state of the art, but because of oversights that could and should have been avoided. Peck pointed out further that "problems are essentially non-quantitative" and that "solutions are essentially non-numerical".

Geotechnical engineering soon devised monitoring as one of the ways to have a better understanding of potential problems. "By appropriate observations in the early stage of construction (…), reliable information can be obtained concerning the real subsurface conditions, as opposed to those that previously could only be deduced or assumed" (Peck, 1973). Accordingly, field instrumentation became an integral part of the design and construction of many geotechnical structures, which can be defined as structures in which soil or rock is a key component, controlling its performance or conditioning its mode of failure. This approach has allowed that possible shortcomings in design or in construction can be mitigated prior to the development of problems. Ideally, the design has no need to be based on the most unfavourable conditions but, more economically, on the most probable conditions. Such approach has been referred to as the Experimental Method, Design-as-You-Go, Observational Method or, more recently, Interactive Design.

The economical advantages of such methods are clear. Technically they increase the reliability of the engineering solution. The disadvantages are mainly related to contractual issues to accommodate design changes during construction. The basic requirements for its application are the need for contingency plans for any foreseen geotechnical condition, the need to anticipate the response of the geotechnical structure in such a condition and the keen ability to interpret the observed performance or to detect subtle deviations. While the first two requirements are undisputed, the last is normally taken for

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granted and overlooked, in many cases resulting in failure to avoid collapse of structures.

There are technical and non-technical reasons for these failures. Some of the latter were addressed by Peck (1973), in his essay on the importance of non-technical factors on the quality of embankment dams: the “abuser” of the Observational Method is one of them. One of the former is the inadequacy of using the Interactive Design in brittle scenarios (Peck, 1969) in which progressive failure may develop requiring “keen insight into the possible behaviour and the most precise measurements” which may not always be available. It is believed that mere comparisons between measurements and predicted quantities may not always suffice to anticipate problems. Rather, these comparisons should be supplemented by the full understanding of the response of the geotechnical structure at hand and by the adequate assessment of the predictions made in the design.

Geotechnical structures are identified according to their particular response as a function of typical stress paths and of specific failure modes. Within this report, four types of geotechnical structures are contemplated: Foundations, Earth Fills, Supported Excavations and Tunnels, covering ample spectrum of responses. Each of the following sections is named after these structures and includes: typical responses, ways of measuring particular performance, critical evaluation of predicting performance, review of procedures for performance evaluation and discussions on specificities of the Interactive Design application on each type of structure together with some case histories. The tentative organization of contents adopted for each section led to an apparent recurrence, which is fully intentional: it works in what some refer to as ‘spiralling analysis’, in which the reader reviews, from given perspectives, different subjects. The process allows comparative cross examination of contents for different subjects, from the same perspective. A final chapter on Field Instrumentation is included, for reviewing basic requirements for instrumentation planning and instruments selection and for discussing new trends and recent developments.

2 FOUNDATIONS

2.1. *Factors Affecting Prediction*

Foundations can be broadly classified as shallow or deep footings (piles). In general terms, for the geotechnical design of a foundation, two fundamental criteria need to be satisfied:

- the foundation should be able to resist the applied loads with an adequate safety factor to account for variabilities in the applied loads and founding soil or rock conditions;
- the expected total and differential settlements under working loads should be less than the tolerable limits for the structure;

Hence the foundation design process involves predictions of the geotechnical load capacity of the foundation and its settlement under given loads.

Factors that would affect the design of the foundations are presented in many foundation engineering text books (eg. Day, 2006; Tomlinson, 1995; Fang, 1991). Among those, the key factors that will affect the predictions of load capacity and settlement of the foundation can be considered as:

- general geology of the area with particular reference to the main geological formations underlying the site and the possibility of subsidence from mineral extraction or other causes;
- detailed information of the soil and rock strata and groundwater conditions within the zones affected by the foundation bearing pressures and construction operations or any deeper strata affecting the site conditions in any way;
- results of laboratory tests on soil and rock samples appropriate to the particular foundation design or construction problems;
- knowledge on the behaviour of similar foundations founded on similar soil or rock conditions;
- previous history and use of the site including information on any defects or failure of existing or former buildings attributable to foundation conditions in the general area of the site;
- method and quality of foundation construction;
- any special features such as the possibility of earthquakes or climatic factors such as flooding, seasonal swelling and shrinkage, permafrost or soil erosion;
- for marine or river structures, information on tidal ranges and river levels, velocity of tidal and river currents, and other hydrographic and meteorological data.

Monitoring and evaluation has played a major role in foundation design since the beginnings of our foundation engineering discipline. It can be argued that most of the available methods to assess the effects of the above factors would have relied upon monitoring and evaluation of foundations. For example, the factors used in the bearing capacity equation, although they are originally based on plasticity theory, have been modified based on evaluations of model and full scale tests.

The state-of-practice in foundation design is still mostly based on assessing an ultimate bearing pressure and then applying a factor of safety. Settlement of the foundation is assessed but a settlement based design is not routinely adopted as a design method. The factors of safety historically used in foundation design were typically adopted to limit the settlement behaviour of the foundation. Atkinson (2007) states that the typical factor of safety of about 3 used in foundation design is probably to control the settlement by limiting the stresses in the underlying soils to be within the range of relatively linear stress-strain behaviour.

The design of shallow foundations generally appears to be satisfactorily understood, and as such is not the subject of as many technical papers in the recent past when compared to the design and performance of deep or pile foundations. Hence, most of the discussion presented in this section is focused on pile foundations.

2.2. *Aspects Related to Pile Foundation*

Coyle and Castello (1981) presented a summary of various bearing capacity factors suggested by various researchers for the estimation of toe resistance of a pile, which is reproduced as Figure 1. There is a considerable scatter, which highlights the variability in the pile design methods.

The shaft and toe resistances for pile design in sand still rely heavily on empirical correlations (Randolph et al, 2005). The ultimate toe resistance is often considered as the resistance assessed at a maximum pile head displacement of 10% of the equivalent pile diameter, rather than the true ultimate resistance (pile plunging under constant load). Fleming (1992) suggested that for cast-in-situ piles, the mobilized toe resistance is generally only about 15 to 20% of the true ultimate toe resistance.

Many of the standards and design codes still provide specific equations to assess the ultimate load capacity of piles, or at least maximum values that should be adopted for different materials. For example, AASHTO (2004) suggest a maximum ultimate toe resistance of 4.3 MPa for bored piles founded in very dense sands (SPT N > 75). The rationale for this limitation is said to limit the pile toe displacement to a maximum of about 0.05 diameters. However taking this value as the ultimate toe resistance and performing a pile design with a traditional factor of safety approach, without paying attention to the total and differential settlement that can be tolerated by the structure, can lead to a very conservative design. It should be noted that for many structures the total settlement may not be of concern and it is only the differential settlement that would affect the structural performance.

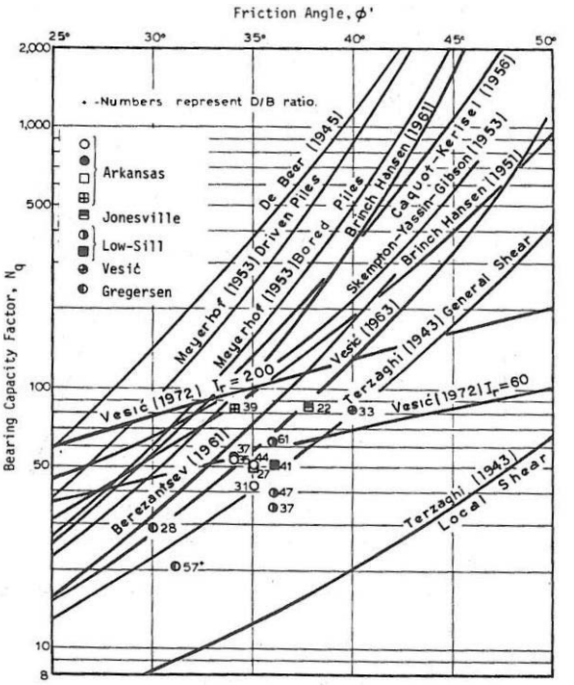


Figure 1. Bearing capacity factor Nq proposed by various researchers (adapted from Coyle and Castello, 1981).

Fellenius (1999) highlighted some inconsistencies which exist in the assessment of ultimate bearing pressure in sands and its effects on pile foundation design. He argues that a bearing capacity failure rarely occurs with respect to pile toes in sands and greater emphasis should be placed on settlement rather than bearing pressure.

A settlement based design for piles founded in rock is also emphasized by Haberfield (2007). He argued that regardless of recent advances in testing, analysis, construction methods and materials, the geotechnical design of foundations in rock for many applications is still dominated by empirical correlations, rules of thumb and traditional values. The application of these "tried and true" methods to current day projects can result in foundations in rock being grossly over-designed with respect to geotechnical performance. The installation process for a pile foundation in rock is often not considered at the design stage and yet it is one of the main factors affecting the performance of the pile foundation. He also noted that there appears to be a widespread reluctance by geotechnical practitioners to move away from the traditional approaches even when provided with prudent alternative design solutions which are logical and defensible and in many cases offer a reduced risk at a reduced cost.

Randolph et al (2005) also highlighted the issue of axial geotechnical load capacity of piles in sand and the urgent need for revision of the current state-of practice. Mandolini et al (2005) also suggested that the conventional capacity based approach is not suited to develop a proper design, and that present codes and standards act as a restraint rather than a stimulus and need to be revised.

A primary driver behind this call for revision in the methods of pile foundation design are the new testing and analytical tools available, such as dynamic load testing and CAPWAP type (Goble et al, 1980) analysis, Statnamic load testing (Middendorp et al. 1992) and testing with Osterberg cells (Osterberg, 1994) . The results obtained from these tests indicate that piles often can carry much higher loads than the ultimate load resistances predicted using present codes and standards. A recent load test in Korea using Osterberg cells on a 2.4 m diameter pile founded at about 56 m depth with about 18 m socket in soft rock has mobilized a gross load capacity of about 236 MN (European Foundations, 2005).

For a settlement based design approach, it is important that the stiffness of the soils and rocks be assessed using both laboratory testing of undisturbed samples and in-situ testing, such as dilatometer, pressuremeter and downhole/crosshole seismic (shear wave velocity) testing. The applied stresses on the founding soils or rocks should also be checked and ideally these stresses should be less than the preconsolidation pressure or bond collapse stress in rocks. This is particularly important in young rocks and considerable settlement due to bond collapse can occur in rock. Figure 2 shows a high pressure consolidation test carried out on a rock sample from a high-rise building development in Dubai where this was one of the controlling aspects in the foundation design. The rock sample tested was calcisiltite with an unconfined compressive strength of about 4 MPa and from a depth of about 84 m.

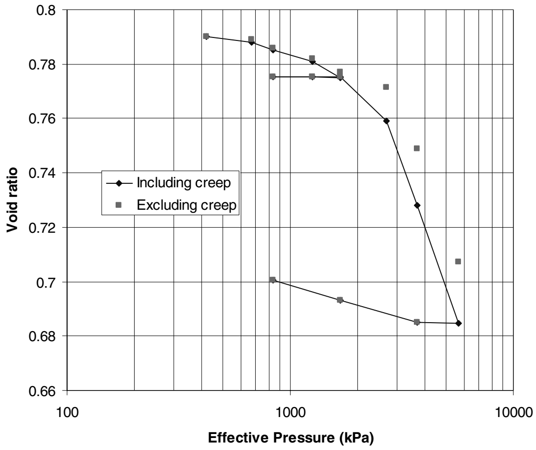


Figure 2. Results of a high pressure consolidation test on a rock sample from Dubai.

2.3. *Evaluation of Prediction*

In the current-state of practice, the methods used to predict the performance of foundation ranges from simple hand calculations based on empirical or semi-empirical equations to complex 3-dimensional finite element analyses. The selected method will depend on the type of foundation and its impact on the existing site conditions such as adjoining settlement sensitive structures.

Evaluation of prediction requires model tests or full scale field testing of the foundations. Model tests or full scale field tests are not routinely carried out for shallow foundations. For deep (pile) foundations, full scale field tests are often carried out. Model tests or full scale field tests will be an important aspect of a project, if:

- there are uncertainties in the design process that require verification;
- the design is pushing the boundaries of accepted past practice and there is a need to satisfy regulatory authorities and critics.

Scale effects are automatically taken into account in model tests in centrifuges. In other model tests, attention should be paid to scale effects and the stress fields, especially in granular materials. Cerato and Lutenegger (2007) investigated the scale effects of shallow foundation model tests. They concluded that behaviour of most model footing tests cannot be directly correlated to the behaviour of full scale tests because of the differences in mean stresses experienced beneath footings of various sizes. They also stated that for a smaller model test in sand to be representative of a full scale test, the model test must be performed in sand that is looser than in the field.

For pile foundations, especially in major projects, the prediction of load capacity and settlement is often evaluated by full scale test piles. An evaluation using a full scale field test should verify most of the factors affecting the prediction as outlined in Section 2.1.

Proper assessment of the test results is also the key in the evaluation process. Many instrumented pile load tests have been misinterpreted, especially with respect to the toe resistance due to the lack of knowledge of residual stresses at the end of pile construction. Analyses of three test piles are presented below to highlight the importance of a proper assessment. The first two test piles were presented by Fellenius (1999), where he critically assessed the pile toe response. The third test pile evaluation is to illustrate an example where the observed results were not as expected, and it was necessary to find a logical explanation for the results to be relied upon.

Test Pile 1

The first test pile is a 2.5 m diameter bored pile founded at about 86 m depth in dense clayey/silty sand for My Thuan Bridge in Vietnam. The pile was load tested with Osterberg cells. The measured load displacement curve for the pile toe is shown in Figure 3. Fellenius argues that a traditional assessment of an ultimate toe resistance by the intercept of two obvious trends on the curve will not be strictly correct for this curve. He argues that the initial stiffer response of the curve is due to residual load that was locked in during pile construction, similar to an unload-reload response. A true hyperbolic toe response could be similar to that shown as an extended curve to the left in Figure 3, if the residual load is considered. The residual toe load of about 10 MN appeared to be equal to the weight of the wet concrete in the pile.

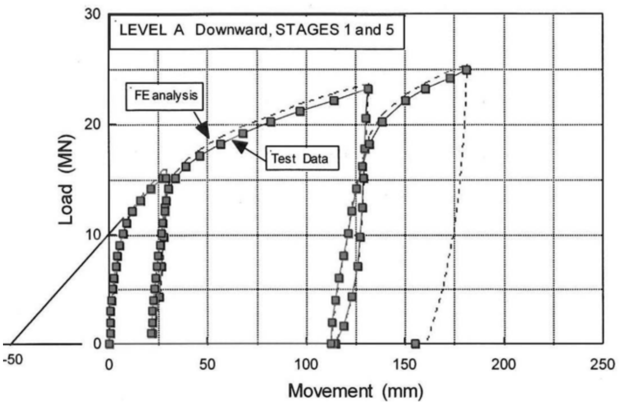


Figure 3. Results of an Osterberg Cell test at toe of pile (adapted from Fellenius, 1999).

A hyperbolic fit to the above toe response may suggest an initial modulus of about 100 MPa and an ultimate toe load of in excess of 30 MN (>6 MPa), if we include the concept of residual load. A case history on the same bridge was presented in Randolph (2003), where he used the same response to assess the behaviour of the pile group foundation of the bridge pier. However, Randolph has adopted an initial stiffness of 450 MPa and an ultimate end bearing resistance of 4.5 MPa, which obviously does not consider the effects of residual load. The toe resistance mobilized is about 5.1 MPa at a toe displacement of about 7% (or about 9% if the precompression movement is included).

Test Pile 2

The second test pile is a 450 mm diameter closed end steel tube pile installed at about 20 m depth in medium dense sands. The pile was subjected to a static load test with a telltale to the pile toe for measuring the pile toe movement. The pile toe load was not measured. The measured pile head and toe movements are shown in Figure 4. Although the load displacement curves would suggest that the ultimate load resistance of the pile has been reached, Fellenius argues that toe resistance is not fully mobilized and that the apparent flat response at the end of the test is due to deterioration of some shaft resistance. His estimated toe load-displacement response is also shown in Figure 4. The assumed shaft resistance of 2000 kN in the assessment of the toe response is based on an adjacent test pile instrumented with strain gauges to measure shaft resistance. The estimated toe load-displacement response also suggests a high residual toe load, which is typically expected in a driven pile.

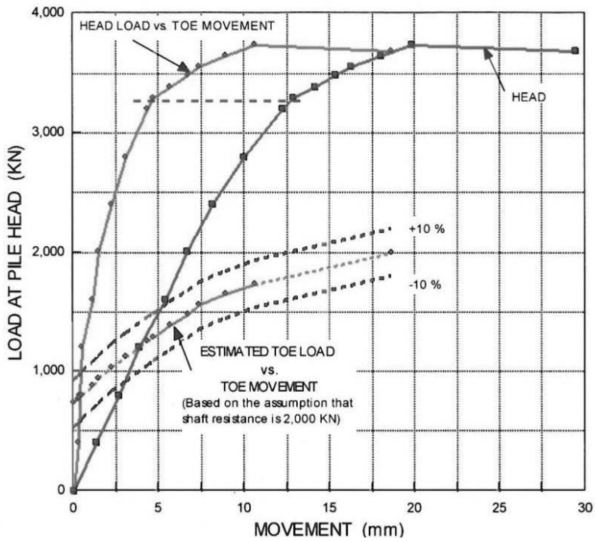


Figure 4. Results of a static load test on a closed end tube pile (adapted from Fellenius, 1999).

Test Pile 3

The third test pile is a 1.05 m diameter bored pile founded at about 28 m depth in mudstone and limestone in Bahrain. The mudstone and limestone rocks were overlain by loose to medium dense sands to about 14 m depth. The pile was instrumented with strain gauges at 5 m depth intervals and was subjected to static load testing. Figure 5 (a) shows the load displacement curve measured at the top of the pile and Figure 5 (b) shows the measured strains at the strain gauges at the various loading stages. The strain gauges in the middle of the pile experienced higher strains than those in the upper part of the pile when the applied load exceeds about 4 MN. The load displacement response at the top of the pile was also softer up to an applied load of 4 MN. Detailed examination of the test set-up revealed that an outer steel casing installed to isolate the upper 7 m of the pile shaft, which is the final cut-off level of the pile, was mistakenly connected to the pilecap constructed for the load testing. It was concluded that up to about 4 MN load, most of the applied load was resisted by the outer steel casing. At higher applied loads, the ultimate geotechnical capacity of the outer steel casing was exceeded and the loads were transferred back on to the mid level of the pile through surrounding soils. The maximum shaft resistance mobilized in the mudstone was about 960 kPa, compared to an ultimate shaft resistance of 500 kPa assumed in the design. No toe resistance was mobilized up to the maximum test load of 20 MN.

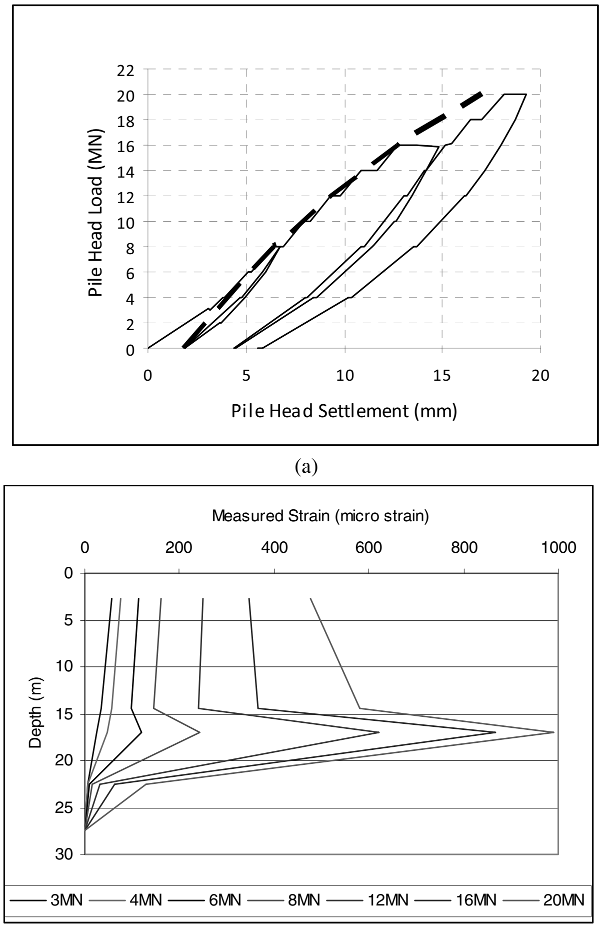


Figure 5. Results of a static pile load test with strain gauges.

2.4. *Evaluation of Performance*

The main reason for monitoring the performance of foundations is to confirm the performance of the structure and the assumptions made in the design.

The key parameter that is required to be monitored to verify the performance of a structure will be the settlement at critical locations, whereby the total and the differential settlement criteria can be assessed. However, a geotechnical assessment of the performance of a foundation will require information on the applied load as well.

As discussed above, for many structures the total settlement may not be of concern and it is only the differential settlement that would affect its structural performance. For example, Figure 6 shows a summary of settlement data of six buildings supported on shallow foundations in Drammen, Norway, over a period up to 20 years, where the total measured settlements were up to about 700 mm and the buildings performed satisfactorily.

Poulos et al (2001) presented tolerable settlement criteria, which are reproduced as Table 1. These criteria were also adopted in the Canadian Foundation Engineering Manual (2006). Poulos et al (op. cit.) also suggested that the values should be used as a guide only for low risk structures, and the allowable values for high risk structures should be assessed individually. Section 4.3 of this paper also provides further information on acceptable deformations to limit the damage to the structures with a complete representation of the deformation pattern rather than a simple angular distortion.

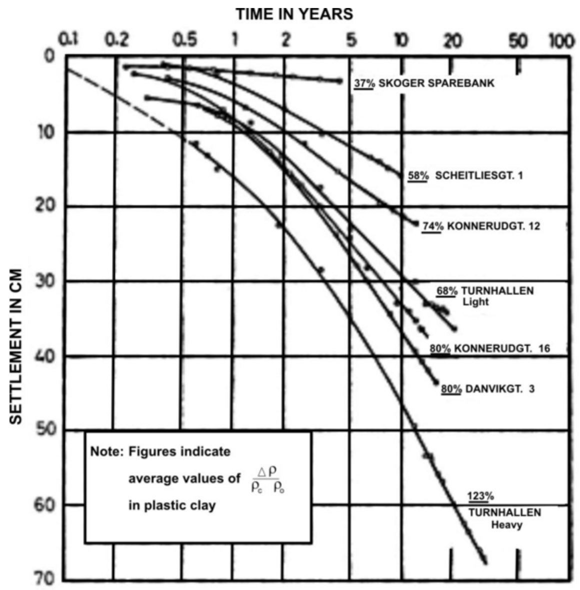


Figure 6. Comparison of settlements of six buildings in Drammen (adapted from Bjerrum, 1967).

Table 1. Tolerable Differential Settlement (after Poulos et al. 2001).

Type of Structure	Type of Damage	Criterion	Limiting Value
Framed buildings and reinforced load bearing walls	Structural damage	Angular distortion	1/150 to 1/250
	Cracking in walls and partitions	Angular distortion	1/500 1/1000 to 1/1400 for end bays
	Visual appearance	Tilt	1/300
	Connection to services	Total settlement	50 to75 mm sands 50 to 135 mm for clays
Tall buildings	Operation of elevators	Tilt	1/1200 to 1/2000
Unreinforced load bearing walls	Cracking by relative sag	Deflection ratio*	1/2500 for wall length/height =1 1/1250 for wall length/height =5
	Cracking by relative hog	Deflection ratio*	1/5000 for wall length/height =1 1/2500 for wall length/height =5
Bridges	Ride quality	Total settlement	100 mm
	Function	Horizontal movement	38 mm (15 in)
	Structural damage	Angular distortion	1/250 for multi span 1/200 for single span

*deflection ratio = maximum relative deflection in a panel/panel length

It should be noted that the total settlement criteria is specified for two cases only – for connection of services in buildings and ride quality in bridges. However, the main issue in these cases will be the differential settlement between the building and the surrounding ground and the differential settlement between the bridge abutment and the approach embankment. In reality, for many projects, this limiting criterion can be overcome by allowing sufficient time to reduce the magnitude of future settlement. In the case of service connections to buildings, this criterion can also be relaxed by using special connections capable of withstanding higher settlements.

In the current-state-of-practice, project specifications often call for load tests on pile foundations to verify the load capacity of individual piles, which is mostly evaluation of prediction rather than of performance. For shallow foundations, specifications may only require assessment and verification of founding materials by a geotechnical engineer.

The real performance of the foundation will depend on the interaction effects between adjacent foundations and interaction with the superstructure. Project specifications often do not call for monitoring of the performance of the foundation. This may be due to the difficulties in establishing monitoring points on foundations which will be often covered during construction and protecting them from accidential damage. Even in cases where monitoring of the foundation settlement is called for, if the observed settlements are within the assumed limits, it is mostly not reported to the geotechnical engineers. An exception to this would be tank foundations, which are often subjected to water load testing prior to commission.

Projects that do come to the fore with settlement measurements are often those with problems and, regrettably, in many cases the information is not published due to issues associated with litigation.

Is should be also noted that normally it would be difficult to change a foundation design when a structure is partly constructed. Therefore an interactive design is not generally feasible with foundations with respect to monitoring and evaluation. This may be another reason for the general conservatism adopted in the foundation design. However, the lessons learnt from the evaluation of the performance can be used for better and improved design in future projects.

2.5 *Case Histories*

Two selected case histories are presented below, where the foundation design of the building was mainly based on settlement.

2.5.1. Twin Towers in Dubai (Poulos and Davids, 2005)

Poulos and Davids (2005) presented a case history of a foundation design for the Emirates Twin Towers in Dubai. The towers are triangular in plan and the office tower, which is the taller of the two, comprised 52 floors and is about 355 m high. The other tower, referred as the hotel tower, is about 305 m high. This case history probably highlights most the aspects discussed above. The subsurface conditions comprised loose to medium dense sand to about 6 m depth, variably cemented sand to about 11 m depth, calcareous sandstone to about 30 m depth, variably cemented sand to about 36 m depth and calcisilite below. The field investigation included pressuremeter and shear wave velocity testing to assess the modulus values in addition to the laboratory testing.

The foundation system consisted of a piled raft comprising 1.2 m diameter piles extending to 40 m to 45 m, with a 1.5 m thick raft.

The predicted load displacement responses were initially evaluated by load testing of 0.9 m diameter test piles. A comparison of the measured and predicted (Class A prediction) load displacement performance of one of the test piles is shown in Figure 7. The measured shaft and end resistances indicate that ultimate pile load capacity will exceed the predicted ultimate pile load capacity of 23 MN by a considerable margin. The axial load distribution within the pile is shown in Figure 8 suggests that minimal toe resistance was mobilized at the predicted ultimate pile load capacity of 23 MN. Poulos and Davids (op. cit.) noted that the shaft friction values used in the prediction were already well in excess of the values commonly used in Dubai. That is, pile design in the region in the past was overly conservative.

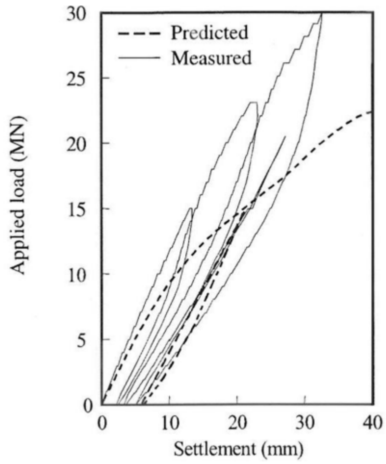


Figure 7. Predicted and measured load settlement behaviour (adapted from Poulos and Davids, 2005).

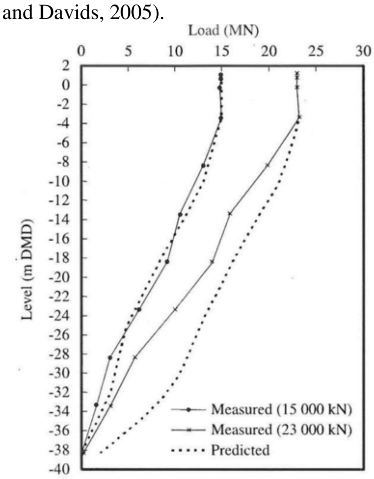


Figure 8. Predicted and measured axial load distribution (adapted from Poulos and Davids, 2005).

Contours of predicted settlement for the hotel tower during the design for serviceability conditions are shown in Figure 9, and indicate a maximum settlement of about 130 mm. The maximum predicted angular distortion for serviceability conditions was about 1/380, which is within the limits indicated in Table 1. Contours of measured settlement when the structure had reached about 70% of its final height are shown in Figure 10. The maximum measured settlement was about 9 mm at 70% completion, which is significantly less than the predicted value of about 50 mm for the same construction stage. Settlement data beyond this construction stage is not available.

Postmortem investigation of the possible reasons for the over prediction of settlement and revised calculations presented by Poulos and Davids, suggests that the modulus value of a weaker layer assumed between about 53 m and 70 m depth has a significant influence on the settlement assessment. The modulus value had to be increased from about 80 MPa to 600 MPa, and the pile interaction factors had to be reduced, to obtain settlements comparable to the measured values. The case history emphasizes the importance of taking proper account of interaction effects in pile group analyses and of allowing for a more realistic distribution of ground stiffness at depth.

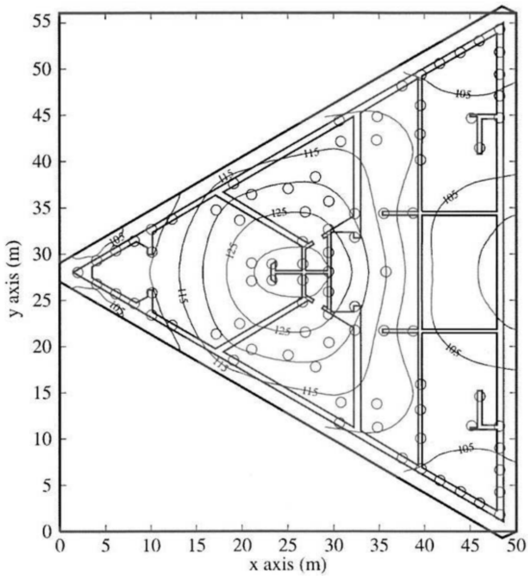


Figure 9. Contours of Predicted Settlement for Hotel Tower (adapted from Poulos and Davids, 2005).

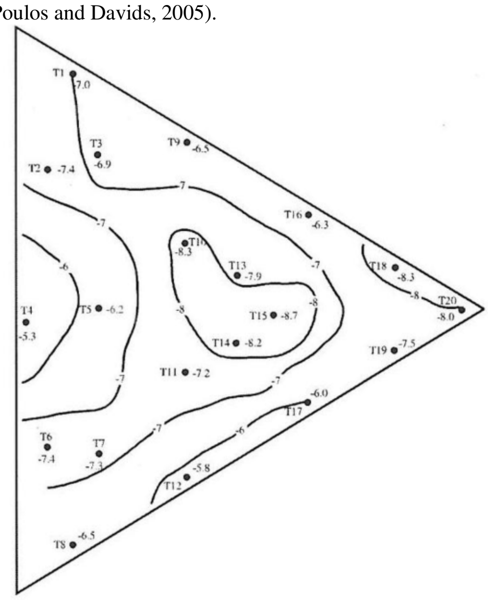


Figure 10. Contours of Measured Settlement for Hotel Tower (adapted from Poulos and Davids, 2005).

2.5.2. Towers in Melbourne

Ervin and Haberfield (2005) presented a case history of a settlement assessment for two residential towers (Towers 2 and 3 of a multi residential tower development) in Melbourne, Australia. These towers had some issues during construction of the foundations. The final design solution adopted was primarily based on settlement considerations.

Tower 2 comprises 20 levels and Tower 3 comprises 31 levels, with a 6 level podium around them. The towers and the podium have a total footprint of about 120 m by 50 m. A plan showing the layout of the towers is shown in Figure 11. Serviceability loads for the core and main columns of Tower 3 are about 60 MN and 25 MN, respectively. Loads for Tower 2 were considerably smaller.

The susbsurface profile at the site can be described as about 2 m of fill overlying 25 m of soft grading to stiff, slightly overconsolidated clay, over 3 m to 7 m of dense to very dense sand (locally known as 'Moray Street Gravel (MSG)') and/or very stiff clay and very dense gravel (locally known as 'Werribee Formation (WF)'), overlying 3 m to 6 m of stiff to very stiff clay (locally known as 'Newport Formation (NF)'), overlying siltstone rock at about 37 m depth.

The tender footing design comprised bored pile footings socketed in siltstone rock for the towers with either driven precast concrete or continuous flight auger piles founding in the MSG or WF unit for the more lightly loaded podium columns. Several piling contractors proposed alternative driven or jacked precast concrete piles installed to practical refusal in siltstone rock for the towers. This alternative scheme was attractive in terms of cost and construction time, but presented a greater risk that piles would not be able penetrate through the MSG and WF units. The piling contractor was confident that they would be able to penetrate these units and carried out preliminary driving tests to confirm this. A driven pile solution was subsequently adopted for the towers.

Towards the end of pile installation, it was revealed that a significant number of piles had not penetrated through the MSG and WF units, as required. The load from the towers would therefore be directly applied to the stiff to very stiff clay of the NF unit, creating a significant risk of settlement of the towers. Detailed settlement analyses were required to assess the likely settlements and to assess any remedial actions that would be required to improve the settlement performance. This required additional field investigation, sampling and oedometer testing of the NF unit.

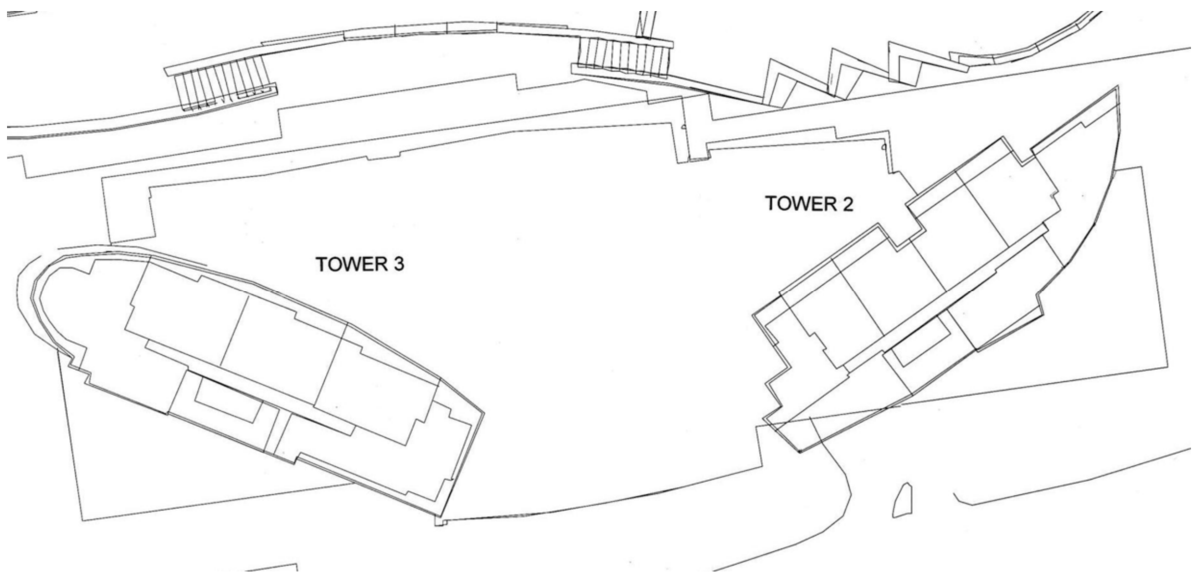


Figure 11. Plan Showing the Footprint of the towers.

The analysis method adopted involved assessment of stresses and settlements in the NF unit using computer programs FLAC (Itasca, 2000) and FLEA (Small and Booker, 1995) and assessing the settlement using spreadsheet calculations. Contours of estimated short term settlements for Towers 2 and 3 are shown in Figures 12 and 13. The long term settlements are estimated to be about 20% higher than the short term settlements.

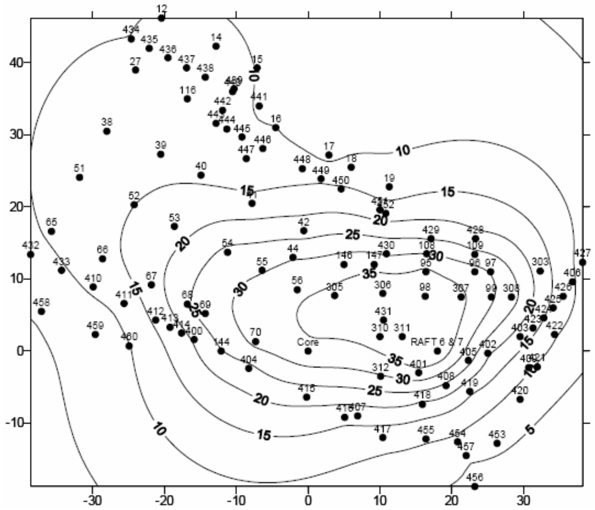


Figure 12. Estimated short term settlement for Tower 2 (adapted from Ervin and Haberfield, 2005).

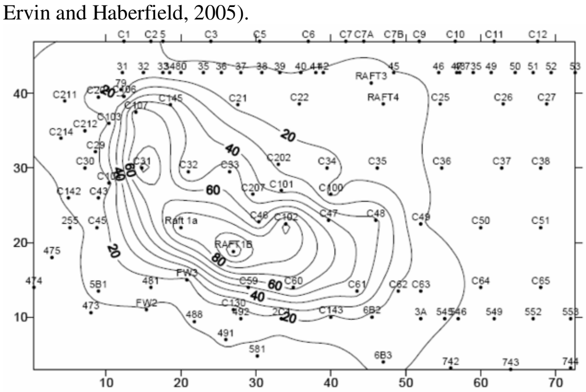


Figure 13. Estimated short term settlement for Tower 3 (adapted from Ervin and Haberfield, 2005).

Based on the estimated total and differential settlements, it was decided that Tower 2 would not require any remedial foundation works. However, the estimated differential settlements for Tower 3 were considered to be excessive, and remedial foundation works were deemed to be necessary. The remedial solution adopted was installation of additional steel H-piles, which were able to be driven through the MSG and WF units to found in the siltstone rock and thereby reducing the vertical stress increase in the NF unit. Test H-piles were installed and subjected to dynamic load testing and CAPWAP analyses to assess load settlement performance. The foundation with proposed H-piles was re-analysed as a composite foundation, with the load redistributions and settlements assessed iteratively to obtain settlement compatibility. Contours of estimated short term settlements for Tower 3 for the inclusion of 76 remedial H-piles at selected locations are shown in Figure 14.

The settlement of selected columns was monitored throughout construction for both towers. Figure 15 shows the contours of measured settlements at the end of construction of Tower 2. These can be compared to the contours shown on Figure 12. The slightly higher measured settlements in the vicinity of Columns 55 and 56 were attributed to heave of the precast piles in the area of up to about 15 mm during driving of remedial H-piles. The precast piles were segmental with compression only joints to manage the stresses during driving. Most of the precast piles were re-struck after the installation of H-piles, but it was not possible at Columns 55 and 56 as the pile cap had already been poured.

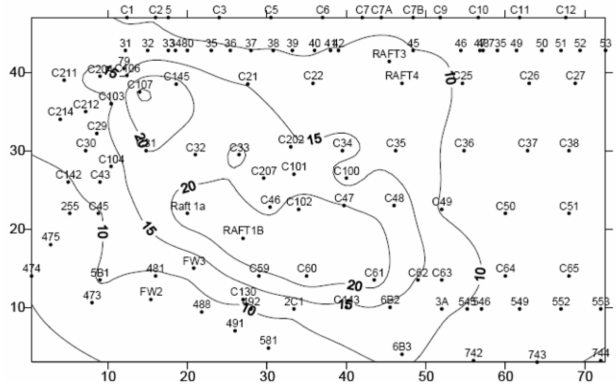


Figure 14. Estimated short term settlement for Tower 3 after installation of remedial piles (adapted from Ervin and Haberfield, 2005).

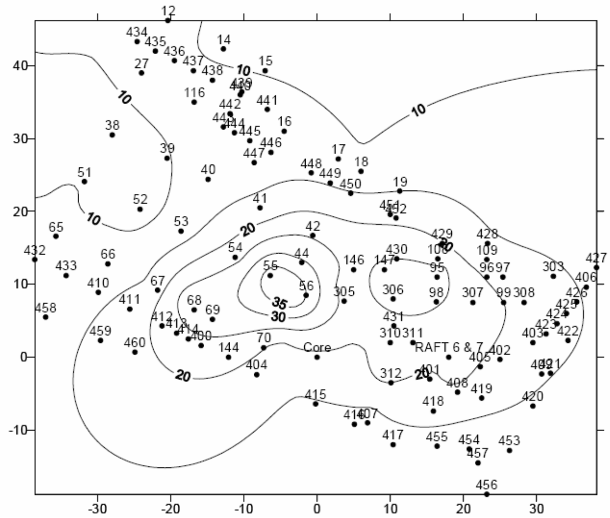


Figure 15. Measured settlement of Tower 2 at the end of construction (adapted from Ervin and Haberfield, 2005).

Monitoring of settlements also occurred at Tower 3. In general, the measured settlements up to the construction of Level 17 of the tower (after which monitoring data was not available) agreed closely with the estimated values.

3 EARTH FILLS

3.1 *Factors Affecting Prediction*

The critical aspects that need to be considered in an earthfill design are:

- selection of the earthfill material to satisfy its intended functions;
- stability of the earthfill and the underlying ground under short and long term conditions;
- settlement of the earthfill and the underlying ground in short and long term.

The design process in an earthfill project will involve prediction of the performance of the earthfill materials for their intended functions, factors of safety for stability, and settlements in the short and long term. The general factors that would affect the above predictions are listed below, which are similar to those outlined in Section 2.1.

- the general geology of the area with particular reference to the main geological formations underlying the site and the possibility of subsidence from mineral extraction or other causes;
- detailed information of the soil and rock strata and groundwater conditions within the zones affected by earthfill construction;
- results of laboratory tests on soil and rock samples appropriate to assess strength, settlement, permeability and dispersion characteristics;
- for earthfill forming a dam to store water or wet tailings, potential changes in groundwater and hydrogeological conditions.
- the engineering properties of available earthfill materials;
- proposed method of construction and equipment;
- knowledge of chemical and physical changes that can occur in the earthfill;
- any special features such as the possibility of earthquakes, or climatic factors such as flooding, seasonal swelling and shrinkage, permafrost or soil erosion;
- for earthfill in marine or river environments, information on tidal ranges and river levels, velocity of tidal and river currents, and other hydrographic and meteorological data.

Embankment dam designs can also be considered to fall into the general description of earthfill design. However the discussions presented in this section are not focused on embankment dams, rather on earthfill on soft ground, where stability and settlement are critical components. For aspects related to dams, reference is made to ICOLD (International Commission on Large Dams) guidelines, ICOLD congress proceedings and many text books on dams (eg. Singh and Varshney, 1995; Fell at el, 2005).

3.1.1 Aspects Related to Earth Fill on Soft Ground

Ladd and DeGroot (2003) define ‘soft ground condition’ as a ground condition comprising predominantly cohesive soils where the applied surface load produces stress that significantly exceeds the preconsolidation stress. Earthfill construction in soft ground requires prediction of the amount and rate of settlement and assessments of stability, especially under undrained conditions. These predictions require some key parameters of soft ground:

- preconsolidation stress or over consolidation ratio;
- compression and re-compression indices;
- initial void ratio;
- in-situ undrained shear strength;
- increase in undrained shear strength with increased effective stress;
- coefficients of vertical and horizontal consolidation;
- secondary compression index.

Most of these parameters are obtained from laboratory testing and some from in-situ tests. Ladd and DeGroot (2003) provided a review of the laboratory testing programs, testing methods and data interpretation techniques required to obtain these parameters. They argue that in spite of significant advances in both knowledge of clay behaviour and field and laboratory testing capabilities, the quality of soft ground site investigation programs has regressed. With respect to laboratory testing and getting good quality samples for testing, they recommend that radiography screening of the tube sample (ASTM D4452) should be carried out more routinely. They also highlight the benefits of constant rate of strain (CRS) testing (Wissa et al. 1971, ASTM D4186) to obtain preconsolidation pressure and compression index values compared to conventional one dimensional consolidation testing with incremental loading (IL). Figure 16 presents a comparison of compression curves obtained from CRS and conventional IL tests. Figure 16 also presents the compression curve using strains at a defined time interval close to the time of end of primary consolidation (EOP) rather than the standard 24 hours. This will be particularly important for soil samples which exhibit considerable creep within the standard 24 hour testing.

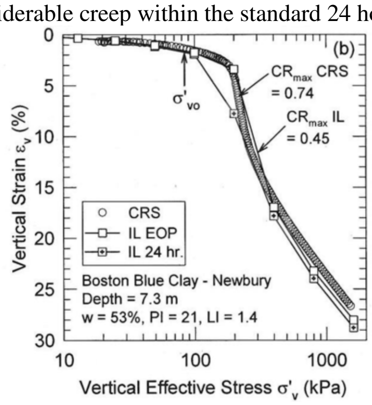


Figure 16. Comparison of compression curves from CRS and IL tests (adapted from Ladd and DeGroot (2003)).

With regard to the assessment of undrained shear strength (S_u) and over consolidation ratio (OCR), Ladd and DeGroot (2003) claim that the field vane test is the most reliable in-situ test. They consider the piezocone testing to be the best in-situ test for soil profiling, however less reliable for assessing the S_u and OCR mainly due to the variations in the coefficient (N_{kt}) used in the empirical correlation. However, it can be used, if a reasonable value of N_{kt} is established from prior experience or using comparison with field vane test. There are however some contradicting views expressed in the literature. For example, based on back analyses of two full scale failures, Long and O’Riordan (2001) concluded that the field vane under predicts the strength of clays in Athlone, Ireland and good estimates of strength are given by the piezocone. Karlsrud et al (2005) also argue that piezocones can be used as a reliable tool to assess the undrained shear strength and a more reliable assessment of the undrained shear strength is obtained when correlations based on a pore pressure factor rather than the cone resistance factor N_{kt} are used.

Ladd and DeGroot (2003) also recommend the use of following equation to assess Su:

$$S_u/\sigma'_v = S\,(OCR)^{m} \qquad (1)$$

where S and m are constants and typical values of S and m are presented in Table 2. The above equation was found to be a vey useful tool by many practitioners for assessment of strength increases in the design of staged embankment construction. For most normally consolidated sedimentary clays, the values in Table 2 would suggest a S_u/σ'_v ratio of 0.22. It is important however, the increase in effective stress is confirmed with pore pressure measurements, rather than estimated based on degree of consolidation assessed from settlements.

Table 2. Typical values of S and m for estimating S_u (after Ladd and DeGroot (2003)).

Soil Description	S	m
Sensitive cemented marine clays (PI < 30% and LI > 1.5)	0.20	1.00
Homogeneous CL and CH sedimentary clays of low to moderate sensitivity (PI = 20 to 80%) (No shells or sand lenses/layers)	0.22	0.80
Northeastern US varved clays	0.16	0.75
Sedimentary deposits of silts and organic soils and clays with shells (excludes peat)	0.25	0.80

Vertical and horizontal coefficients of consolidation (c_v and c_h) can be obtained from laboratory consolidation and field dissipation tests during piezocone testing. Although it is not true for all clay deposits, there are many case histories where the c_v values obtained from field consolidation performance are much higher than the laboratory estimated values. Leroueil (1988) compiled data from 16 embankments and found that, on average, c_v values obtained from field settlement data were about 20 times higher than the laboratory values. It is possible the faster field consolidation may be due to shorter drainage paths rather than higher c_v. For example, field settlement performance of a soft clay deposit, known as the Coode Island Silt, in Melbourne, Australia also indicate a field c_v of about 20 times higher than the laboratory c_v, when an effective drainage path is assumed to be half of the thickness of the deposit. Closer examination of continuous samples of the deposit revealed the presence of very thin fine sand layers at various depth intervals. These very thin sand layers are difficult to detect during drilling of boreholes or even in piezocone testing. However in alluvial deposits of soft clays, it is highly possible that these thin sandy layers could occur in the natural deposition process. This indicates that the apparent increase in the field c_v may actually be due to a reduction in the effective drainage path rather than an increase in c_v. An increase in the field c_v by a factor of 20 is equivalent to a reduction in drainage path by a factor of about 4.5.

Another aspect with earthfill construction in soft ground is the need ground improvement, especially for projects with large thickness of earthfill. Discussions on ground improvement methods are presented in Mitchell (1981), Terashi and Juran (2000) and Munfakh and Wylie, (2000). Typical methods of ground improvement adopted in soft ground include staged construction, surcharging, acceleration consolidation by preloading with and without artificial drainage, soil replacement, deep soil mixing and the use of stone columns or pile support.

3.2. *Evaluation of Prediction*

In the current state of practice, the design process or the prediction of performance will mostly involve some form of numerical modelling. Examples are stress deformation analyses using finite element or finite difference software, slope stability analyses using software with limit equilibrium methods and settlement assessments using spreadsheets.

The modeling processes however have limitations with respect to the ability to model the variabilities within material layers/zones and the real material behaviour. Natural soil deposits have inherent variabilities and other associated uncertainties and therefore the field performance could be different from the prediction.

If a true evaluation of prediction is required, an instrumented trial of the earthfill will need to be constructed. Instrumented trial embankments have played a major role in quantifying and reducing the risk associated with these variabilities and uncertainties. Instrumented trial embankments are particularly important when there is high variability in soil conditions and/or there is no previous experience. The observed performance of the trial embankment will need to be checked against prediction. If the performance is different, then the models used for the prediction will need to be revised and recalibrated with back analysis of the observed data.

There are a number of projects reported in the literature which involved trial embankments. One of the first applications of instrumented trial embankment is given in Ringeling (1936) in the first International Conference on Soil Mechanics and Foundation Engineering. Bishop and Green (1973) presented a review of the development and use of trial embankments. They have listed the type of instruments used in earthfill projects, which have not changed significantly over the last 35 years, although the electronic components used in the instruments and accuracy of the data have been improved. The monitoring instruments used in earth fill projects are summarised in Table 3.

Table 3. Summary of monitoring instruments used in earth fill projects.

Instrument	Monitoring Parameter
Piezometer (standpipe, hydraulic, pneumatic, vibrating wire)	Pore pressure
Surface monuments	Surface settlement, horizontal movement
Settlement plate	Settlement at specific depth
Settlement profiler	Settlement along a section at a selected depth
Extensometer (Rod, magnetic, wire, tape)	Settlement at selected depths, vertical strains
Inclinometer	Horizontal movement with depth, tilt
Strain meter	Horizontal movement at selected locations, horizontal strain
Pressure cell	Earth pressure
Tilt meter	Tilt

One of the largest trial embankments in the world, involving the placement of 1 million m^3 of earthfill was constructed, for the for Morwell River Diversion project at the Yallourn coal mine in Victoria, Australia. The results from this project are further discussed in Section 3.5.1 as a case history.

3.3 *Interactive Design*

As earthfill construction on soft ground usually is carried out over an extended period of time, interactive design (or observational design method) may also be used, utilizing the performance of the early stages of the earthfill construction to decide future construction activity. Staged embankment construction is a good example of this. In the interactive design approach, it is important that the stakeholders are made aware of the potential adverse design changes that may occur and the associated cost and time implications.

Interactive design for embankments is a similar process to the evaluation of performance, but the performance will be continuously checked and the design tools and models are continuously updated and re-calibrated, as required whilst the future performance is re-assessed.

With interactive design for the assessment of rate and degree of consolidation, observational methods such as Asaoka (1978) and Tan and Chew (1996) have been used quite extensively. As discussed in Section 3.1.1, for the assessment of increase in undrained shear strength, the equation given by Ladd and Degroot (2003) is very useful.

An example of the interactive design approach was presented in Srithar and Ervin (2000), and involved staged construction of a 7.4 m high approach embankment on about 18 m thick soft clay deposit for the My Thuan Bridge project in Vietnam. As the measured strength increase after Stage 1 construction was higher than that expected during design, it was possible to construct the next two stages of construction as a single stage, which resulted in savings in construction time and cost. The embankment construction stages are shown in Figure 17, the measured settlements at selected monitoring points are shown in Figure 18, and the comparison of CPT tip resistances at the start of the project and at the end Stage 1 consolidation is shown in Figure 19. The CPT results were used to assess the strength increase.

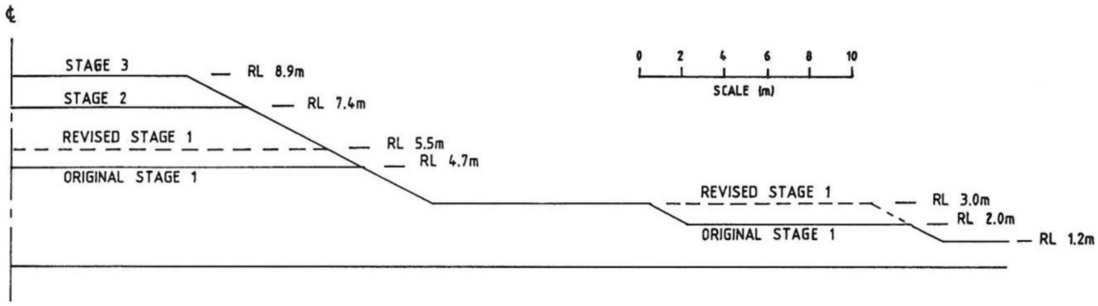


Figure 17. Embankment construction stages –My Thuan Bridge, Vietnam (adapted from Srithar and Ervin (2000)).

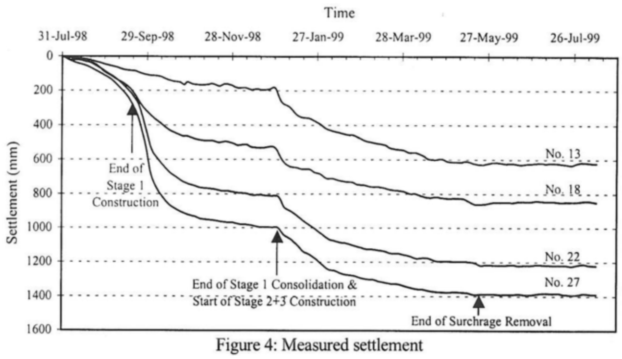


Figure 18. Measured settlements–My Thuan Bridge, Vietnam (adapted from Srithar and Ervin (2000)).

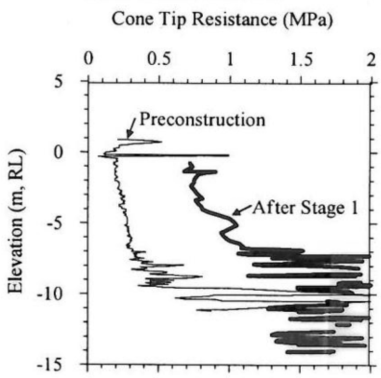


Figure 19. Comparison of CPT tip resistances –My Thuan Bridge, Vietnam (adapted from Srithar and Ervin (2000)).

In an another example of interactive design, presented by Ozcoban et al (2007), it was necessary to delay the earthfill placement and even removal of some of the earthfill due to higher observed pore pressures than predicted and the associated stability concerns. The design then adopted the installation of sand drains in various areas to increase the rate of pore pressure dissipation.

3.4. *Evaluation of Performance.*

Ervin (1988) states that fundamental to effective evaluation of any observed geotechnical performance is a thorough understanding of the geotechnical engineering principles pertinent to the particular problem. It will be human nature to try to fit the data to a preconceived model or behaviour. However, if a satisfactory fit is not achieved of the all the observed data that is considered reliable, both the model and the relevance of the data to the particular model need careful consideration. Any attempt to massage the soil model to suit the data obtained or to dismiss data which does not fit the chosen model implies a lack of confidence in either the data or the design model and the real behaviour may be quite different. Such lack of objectiveness should be assiduously avoided.

With respect to earthfill on soft ground, the key parameters for assessing the performance will be stability and settlement. An acceptable performance with respect to stability is generally confirmed by decreasing trends of observed settlement and lateral movements and acceptable values of pore pressures. Typical minimum factors of safety adopted in earth fill construction in soft clay, when limit equilibrium methods such as simplified Bishop are used to predict the factor of safety, are presented in Table 4.

Table 4. Typical factors of safety adopted for earthfill embankments in soft clay.

Condition	Soft clay strength input	Minimum factor of safety
Long term	Undrained shear strength expected in long term after consolidation	1.5
Short term	Undrained shear strength prior to construction	1.3

The above factors of safety values should be considered as a guide only. Lower or higher values can be adopted, which may relate to local experience, reliability of the selected input parameters, sophistication in the stability assessment and the risk associated with the project. For major projects, a suitable minimum factor of safety may need to be adopted based on reliability analysis, such as Christian et al (1994).

Table 5 presents a summary of typical settlement criteria adopted for various projects involving earthfill in soft ground. These criteria should be established during the design process. Again the limiting values presented in the table should be considered as a guide only. Lower or higher values can be adopted depending on the local experience, quality control adopted during construction and the risk associated with the project.

Table 5. Typical settlement criteria adopted for earthfill projects.

Structure on earth fill	Criterion	Limiting Value
Road including bridge approach road	Long term total settlement after road construction	50 mm
	Differential settlement	20 mm over 5 m
Bridge abutment	Lateral movement after footing installation	25 mm (not applicable, if footing is designed for lateral soil movement)
Building on shallow foundations	Angular distortion	1/150 to 1/250 (will depend on the type of building)
Building - Service connection through earthfill	Differential settlement	50 mm (will depend on type of connection)
Buried service pipe	Angular distortion	1/200 (will depend on the type of pipe)
Clay liner	Angular distortion	1/5 to 1/200 (will depend o the type of clay)

3.5. *Case Histories.*

3.5.1 Morwell River Diversion

Srithar and Ervin (2007) presented a case history of a large earthfill project for the diversion of the Morwell River in Victoria, Australia. The project involved construction of a 70 m wide, 3.5 km long river diversion channel mostly through a loosely backfilled brown coal mine pit in Latrobe Valley of Victoria. The total volume of earthworks was about 13 million m^3. The maximum height of the embankment was about 60 m, with up to 30 m high built over about 50 m of uncontrolled loosely dumped fill (derived from overburden stripping for coal mine operations) comprising a mixture of materials including clays with very high variability. Some of the key design issues of the project were stability of the embankment, magnitude and rate of settlement, potential strains in the clay liner and construction of culverts for conveyors serving on-going mining operations. A perspective view of the diversion embankment is shown in Figure 20.

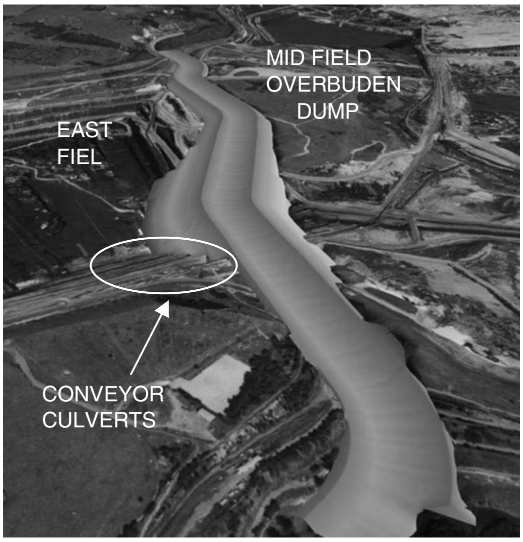


Figure 20. Perspective view of river diversion embankment.

The dumped materials generally comprise clays with layers of sands and gravels. The consistency of the clays was soft to firm to about 30 m depth and stiff below somewhat typical of a slightly overconsolidated natural clay deposit. The relative density of the sand and gravel layers generally ranged from loose to medium dense. Typical profiles of cone tip resistance and friction ratio from a cone penetration test (CPT) are shown in Figure 21. Profiles of CPT tip resistance from a few CPTs are shown in Figure 22 to illustrate the variability. The clays were mostly of medium to high plasticity. The Liquid Limit was in the range of 25% to 65% and the Plasticity Index was in therange of 15% to 45%, initial void ratio was in the range of 0.54 to 0.76 and the compression index was in the range of 0.10 to 0.16.

Because of the high variability in the dumped fill materials and associated risks, a trial embankment involving the placement of about 1 million m^3 of earth fill (about 400 m long, 200 mwide and up to 15 m high) was constructed. The area of the trial embankment was chosen to be within the footprint of the main diversion embankment so that it could be part of the main embankment. A cross section of the main and trial embakments is shown in Figure 23.

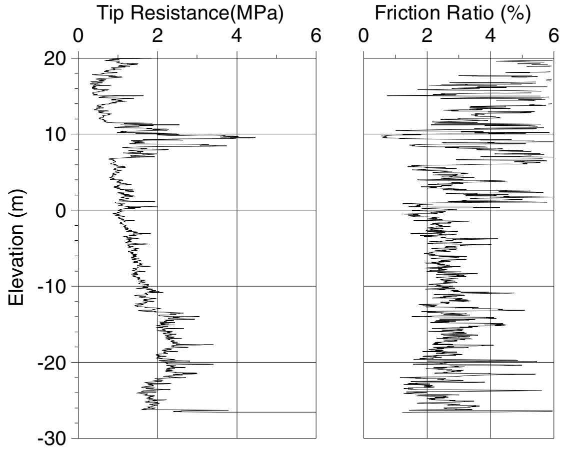


Figure 21. Typical CPT results in dumped fill materials.

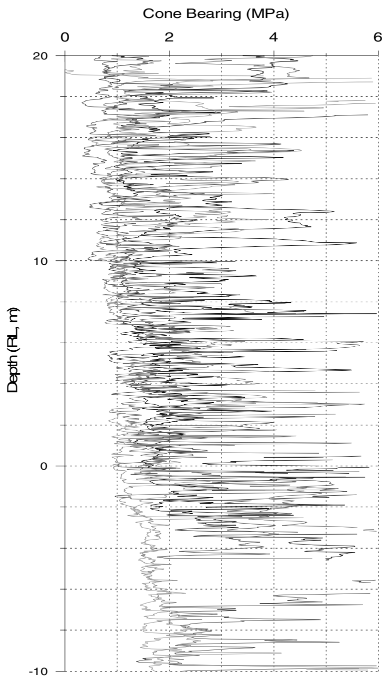


Figure 22. CPT tip resistance profiles in the dumped fill from 6 CPTs.

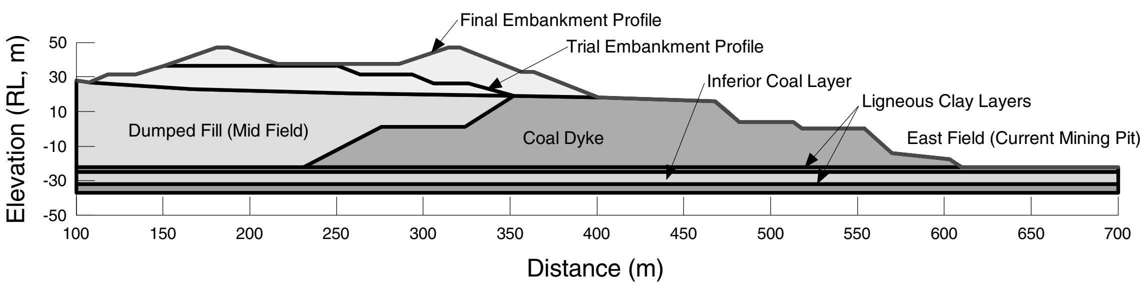


Figure 23. Cross section of Morwell River diversion embankment.

The trial embankment area was instrumented with vibrating wire piezometers, magnetic extensometers, settlement plates and surface settlement pins. Additional CPT tests were carried out one year after the construction of the trial embankment to assess strength increase in the dumped fill materials.

A plot of typical settlements measured in settlement plates is shown in Figure 24. Typical results of magnetic extensometers, which provide settlement within the dumped fill materials relative to a base magnet anchored in the natural materials about 5 m below the base of the dumped fill, are also shown in Figure 24. Typical excess pore pressures measured in vibrating wire piezometers at three different depths are shown in Figure 25. A comparison of cone tip resistances in the dumped fill before and one year after the trial embankment construction is shown in Figure 26.

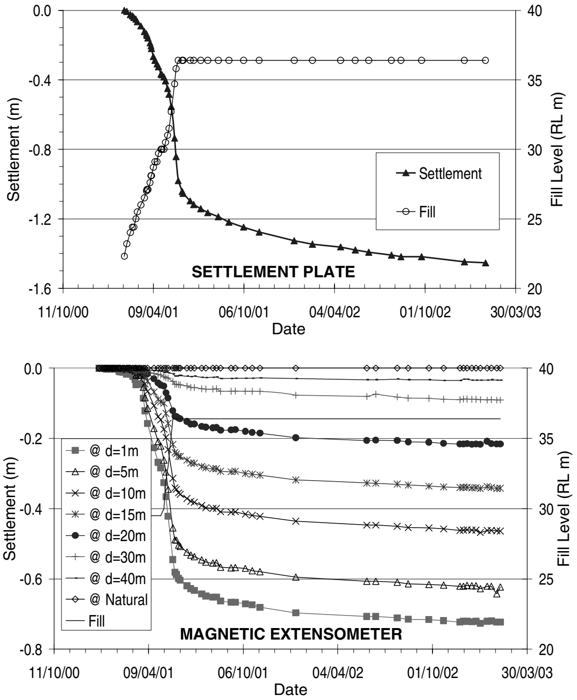


Figure 24. Typical settlements measured in settlement plates and magnetic extensometers.

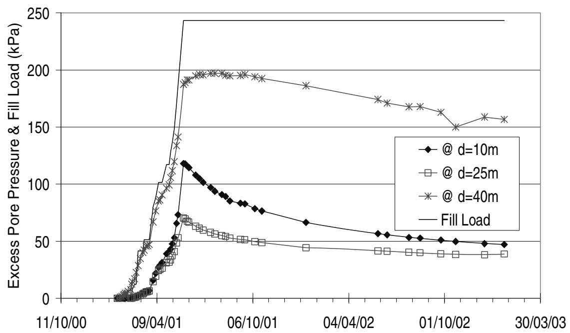


Figure 25. Typical excess pore pressures measured in vibrating wire piezometers.

The monitoring results of the trial embankment, site observations, subsequent investigations and back analyses revealed the following:

- settlement up to about 2.5 m could occur in the areas of dumped fill under the proposed diversion embankment;
- about 40% of the above settlement is due to the compression of the natural materials underlying the dumped fill. This was not taken into account in the preliminary design stage and was one the key finding from the trial embankment. This was assessed to be mainly due to the presence of layers of brown coal in the underlying natural materials. Brown coal is a heavily overconsolidated material, but the moisture content is about 100% to 150% and its re-compression index is high (about 0.035);
- about 90% of the settlements would occur within about 1 year of fill placement. This rate of settlement was faster than initially anticipated. This was another key finding;
- the degree of observed settlement did not match the degree of observed pore pressure dissipation in the overburden dumped materials. The degree of pore pressure dissipation in 1 year was about 70% to 80% in the piezometers installed within upper 25 m the dumped fill and about 25% below. This pore pressure behaviour of the deeper piezometers did not agree with the settlement behaviour of the soils at similar depths observed in extensometers. A similar phenomenon was also reported in Mitchell (1986) in soft clays and was attributed to collapsing of the soil structure, resulting in lower permeability of the soil retarding the dissipation of the pore pressure;
- the strength increase in the dumped materials is related to the increase in effective stress. The assessed average increase in undrained shear strength was about 25 kPa in the top 25 m of the dumped fill and about 5 kPa below. This was another key finding. This also highlights the aspect that the strength increase will depend on the dissipation of pore pressure (or increase in effective stress) regardless of the high degree of consolidation assessed from settlements.

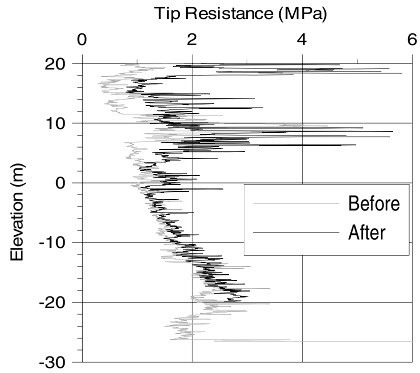


Figure 26. Comparison of CPT tip resistance profile before and after trial embankment.

The main embankment design was revised based on the results observed in the trial embankment and was constructed in 3 stages over a three year period. Slow dissipation of excess pore pressure below 25 m depth and the associated lower strength increase required the placement of a minor toe berm in one area to improve stability. The main embankment was instrumented with 25 settlement plates, 3 magnetic extensometers, 10 inclinometers, more than 60 surface settlement monuments, 30 standpipes and 70 vibrating wire piezometers. The plots of settlements at various settlement plates installed in the main embankment to the completion of the project are shown in Figure 27. The maximum measured settlement was close the maximum predicted value of 2.5 m.

The project required the construction of four culverts for conveyors, with fills up to 45 m above the base of the culverts and with varying foundation conditions. The culverts were designed for maximum settlements up to 2 m and associated differential settlements. The culvert segments were designed with sufficient articulation to accommodate the predicted differential settlement, and were constructed with a camber mirror imaging the predicted settlement profile to result in an acceptable final grade. The culverts were constructed parallel to each other at about 50 m spacing, but at different levels. Design profiles of the culverts are shown in Figure 28.

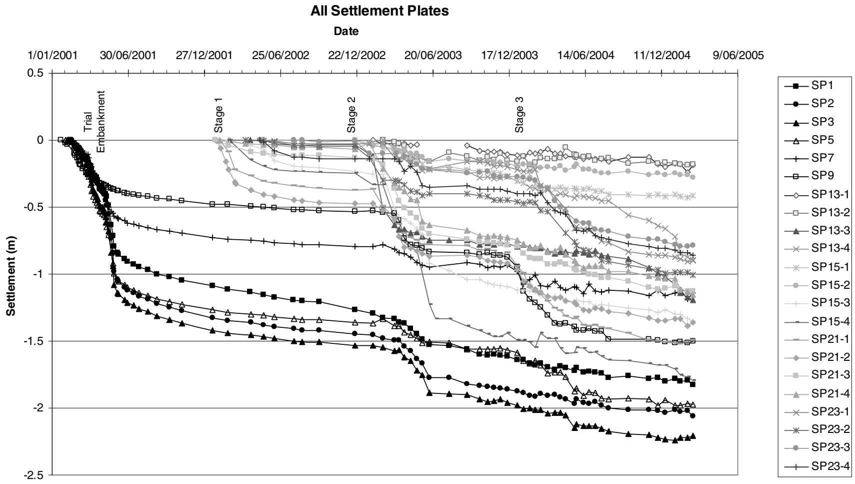


Figure 27. Settlements measured in settlement plates.

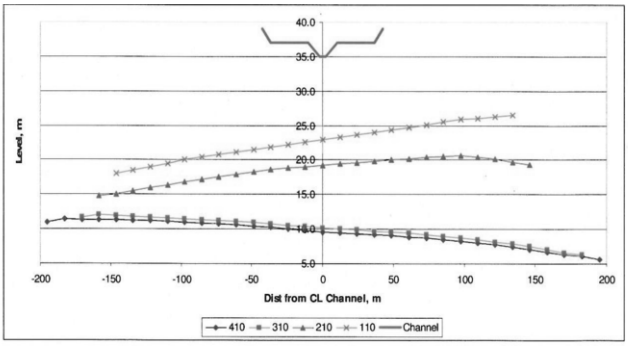


Figure 28. Design profiles of the conveyor culverts (adapted from Jenkins and Lawson, 2007).

Jenkins and Lawson (2007) present an assessment of the observed and predicted culvert settlements and the possible reasons for the variations. The predicted and measured settlements along the four culverts are shown in Figure 29. The measured settlements were within the predicted range of settlements, allowing for the variability in the underlying materials. Possible reasons for the variability in the measured settlements compared to the average predicted settlements were assessed to be:

- placement of fill over lower level conveyors (410 and 310) might have caused some settlement in the area of upper level conveyors (210 and 110) before they were installed and monitored;
- variations in the construction sequence adopted in modeling and actual construction;
- inherent variability in the underlying materials.

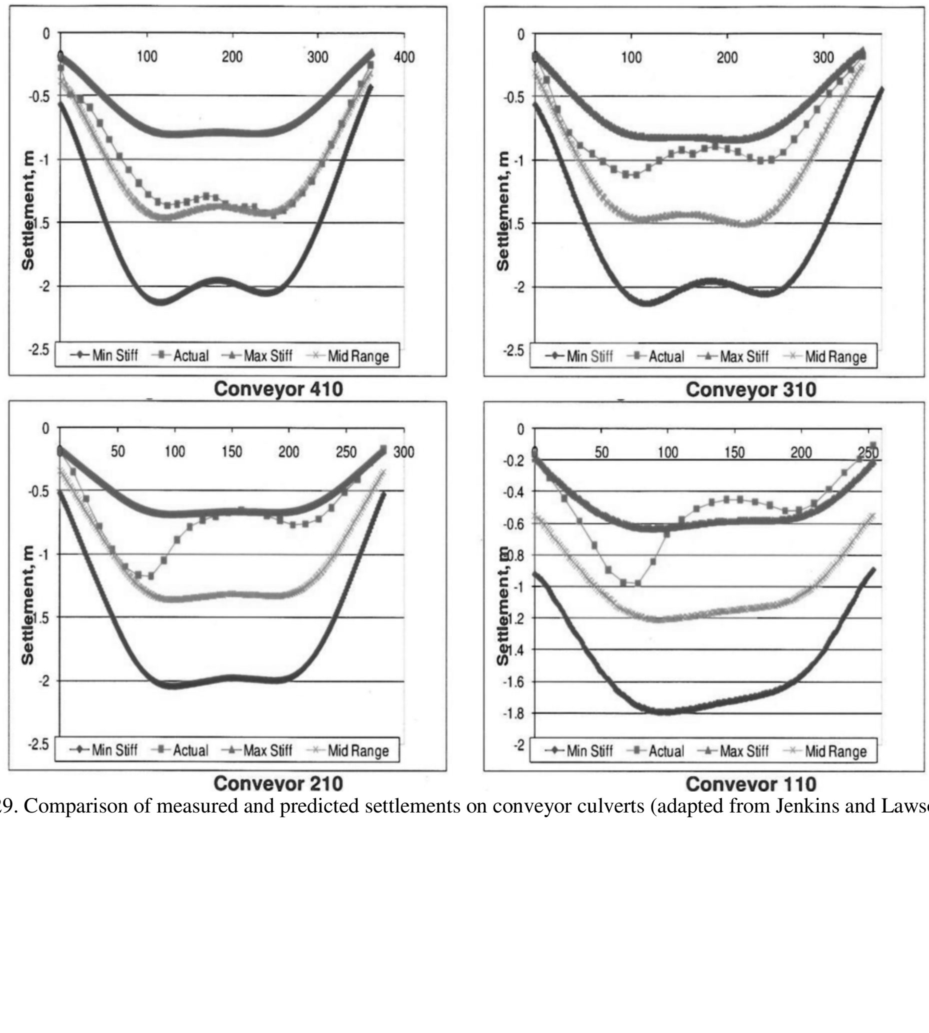


Figure 29. Comparison of measured and predicted settlements on conveyor culverts (adapted from Jenkins and Lawson, 2007).

3.5.2 *Alibey Dam in Turkey (Ozcoban et al (2007)*

Ozcoban et al (2007) presented a unique case history of Ailbey Dam construction near Istanbul in Turkey, where the construction spanned over a period of over 15 years (1967 to 1983) and the monitoring data span over 25 years. Ailbey dam is an earthfill dam with a maximum fill height of about 28 m, constructed over 30 m thick soft alluvial sediments in stages. The total volume of dam earthfill was about 2 million m^3. Plan and cross section of the dam are shown in Figures 30 and 31. The dam was instrumented with 16 settlement plates, 88 piezometers (43 hydraulic, 10 electrical, 20 Bishop pneumatic and 15 Bishop hydraulic) and 5 inclinometers.

Selected areas during initial stages of construction were used as test embankments. As the test fill height reached about 6 m, higher excess pore pressures were measured and consequently 1 m of fill was removed and the filling was delayed for one year due to stability concerns. Sand drains were then installed to selected areas to increase the pore pressure dissipation.

Ozcoban et al (2007) presented an evaluation of the methods of settlement prediction in a historical context, starting from the simplified calculations based on one dimensional consolidation theory at the start of the project in the late sixties, to the use of modern finite element software. They have noted that the rate of consolidation (or coefficient of consolidation) was the key factor in prediction of behaviour similar to that observed. They found that the coefficient of consolidation obtained using Asaoka (1978) method based on observed settlements would give reasonable predictions. The laboratory coefficients of consolidation were found to be about 25 times smaller than those obtained from field settlement measurements.

Observed behaviour was back analysed with the computer program PLAXIS using the soft soil stress-strain model in the program, which is based on modified Cam Clay model. An additional field and laboratory investigation program was carried out in 1996 to verify the material parameters, which included CRS consolidation tests. The compressibility and permeability parameters obtained from 1966 and 1996 investigations are not significantly different.

Comparisons of observed and computed behaviours are shown in Figures 32 and 33. The recorded and computed pore water pressures compare well when a reference water level of +12 m is used. The dam was used to store water long before reaching the final height and water level of +12 m corresponds to the average water level in the reservoir.

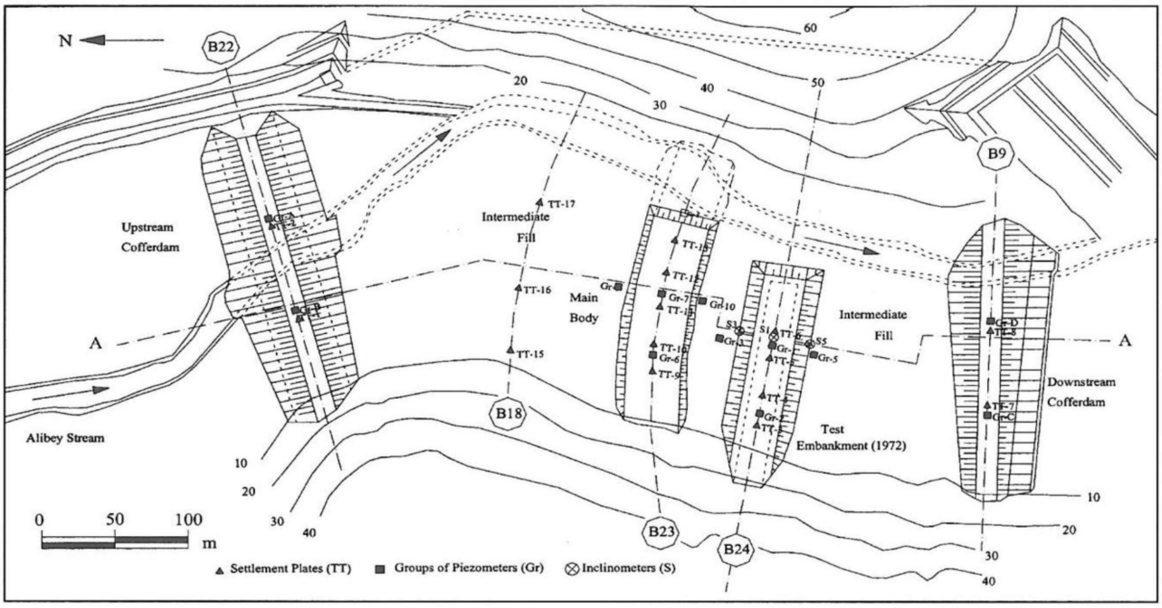


Figure 30. Plan view of Alibey dam (adapted from Ozcoban et al, 2007).

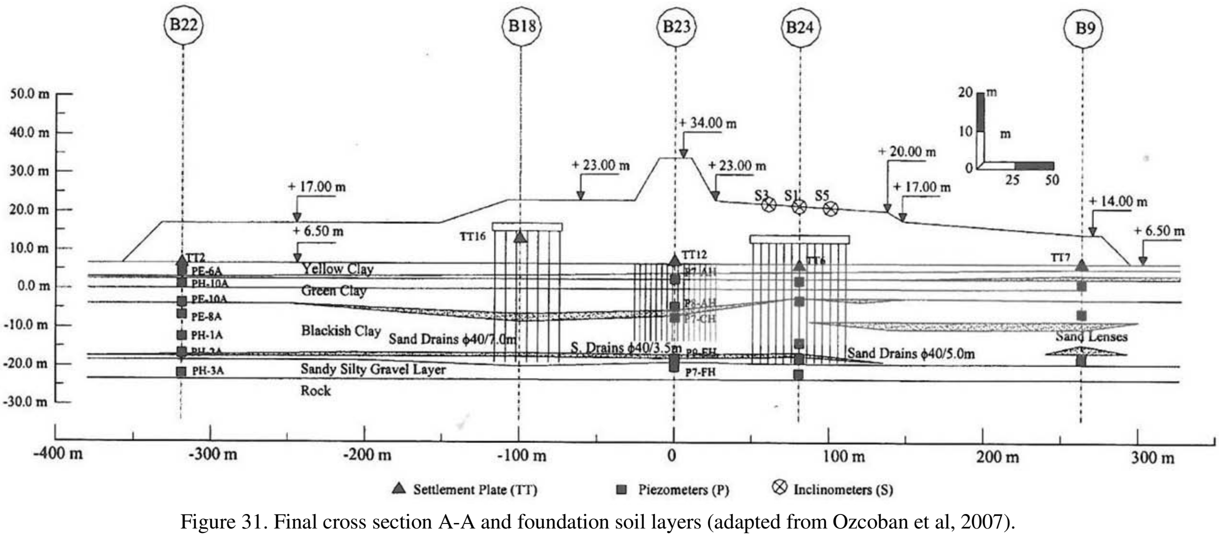


Figure 31. Final cross section A-A and foundation soil layers (adapted from Ozcoban et al, 2007).

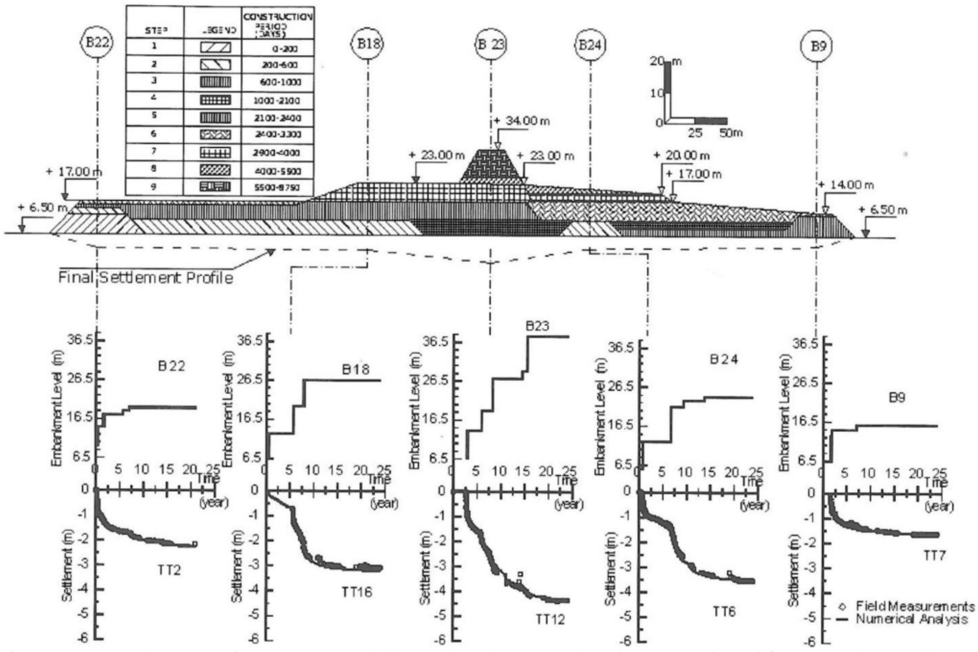


Figure 32. Staged construction program and observed and computed settlements (adapted from Ozcoban et al, 2007).

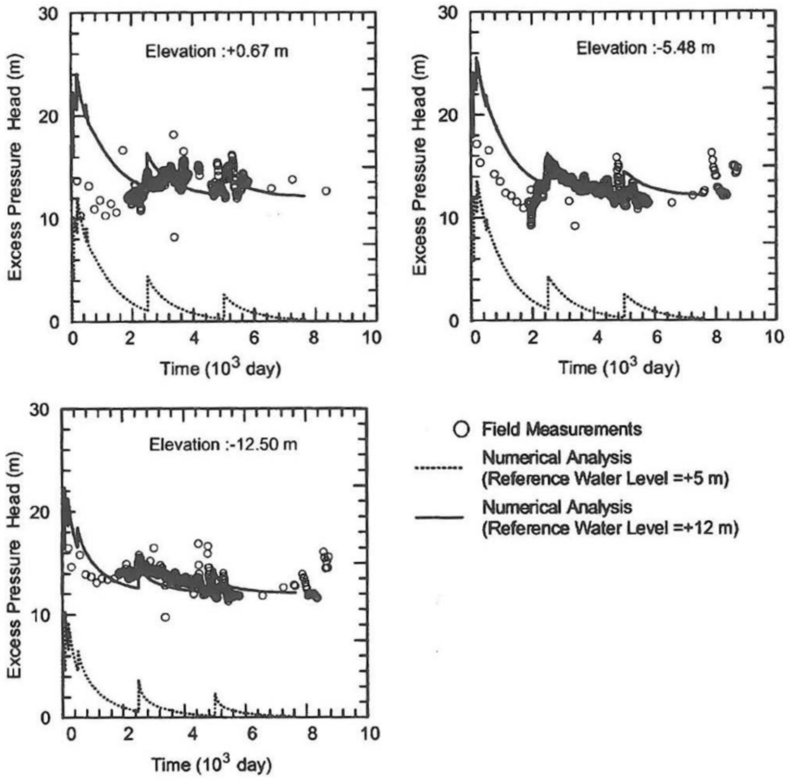


Figure 33. Observed and computed excess pore pressure heads at Section B22 (adapted from Ozcoban et al, 2007).

Some of the key learnings from the project were:

- in staged embankment construction in soft clays, pore water pressure build up is very important for stability;
- with proper instrumentation and careful assessment of collected data, field construction rates can be adjusted for stable earthfill placement in soft clays;
- standard subsurface field and laboratory investigation programs may not provide realistic parameters to assess the rate of consolidation.

4 SUPPORTED EXCAVATIONS

4.1. *Factors affecting stability loads and deformations*

4.1.1 Stability

Assuming that the support system is properly designed, the geotechnical stability of supported excavations is in general governed by classical bottom heave mechanisms in soft to medium stiff clays and hydraulic uplift stability in frictional soils. In stiff clays, or sands above the water table, stability is in general not an issue as long as struts or anchors are properly designed to take the anticipated loads. For soil anchored walls the stability of a potential failure surface extending behind the anchor bodies must be ensured.

In soft to medium stiff clays the bottom heave stability can be well predicted by a modified Bjerrum&Eide (1956) approach as proposed by Karlsrud and Andresen (2008). The approach accounts for the effect of wall penetration below the bottom of the excavation and limited depth to firm strata, and was verified for a variety of 2D and 3D cases.

According to Karlsrud and Andresen (2008) the bottom heave safety factor can be found from eq. (2) and (3). Figure 34 defines the geometry and Figure 35 presents the dependency of stability number, N_c, on the geometry.

$$F = (N_c s_{ub} + 2 s_{uT}\, z_T/B_{cr}) / (\gamma H + q) \quad (2)$$
$$F = 0.94 N_c s_{ub} / (\gamma H + q - p_{My}(z_T / z_{cr})) \quad (3)$$

$p_{My} = (2M_Y - \sigma_{ha} z_S^2)/(z_T^2 + 2 z_S z_T)$

s_{ub} = average strength in bottom heave failure zone below the tip of the wall

s_{uDw}= average strength over height D_w of the wall

s_{uT} = average strength over toe depth z_T of the assumed rigid wall, and accounting for possible strength reduction at wall/ clay interface.

σ_{ha} = average horizontal earth pressure on supported side from lowest strut to bottom of excavation.

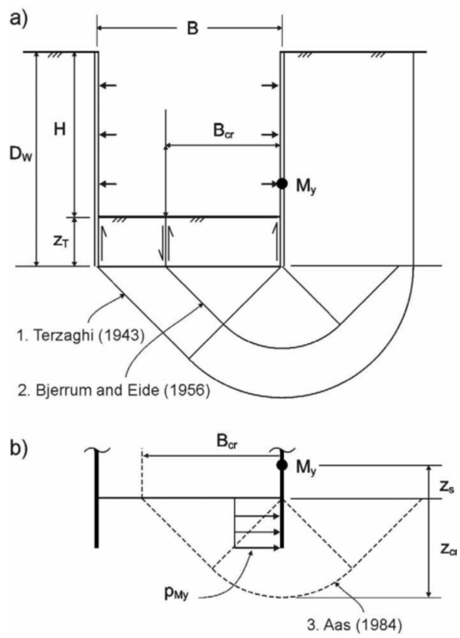


Figure 34. Geometry definitions related to bottom heave stability analyses in clays (from Karlsrud & Andresen, 2008).

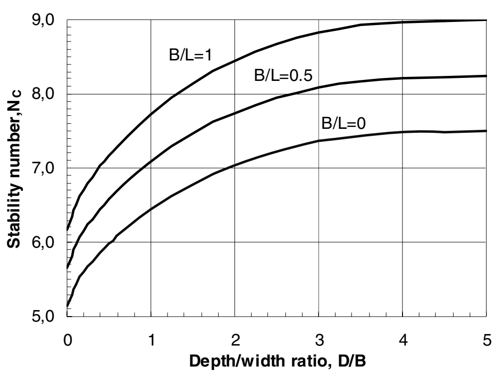


Figure 35. Bottom heave stability chart from Janbu et al (1966) based on the Bjerrum and Eide (1956) approach.

Eq. (2) is valid for the case of an infinitely stiff wall, and (3) for a wall with limited bending moment capacity, M_y.

Karlsrud &Andresen (2008) also presented and discussed the merits of a variety of methods that can be used to improve bottom heave stability as well as limit displacements, some of which will be discussed later. The methods include:

- excavating in sections, thereby increasing the stability number;
- deeply embedded high capacity wall;
- underwater excavation and base slab construction;
- excavating under air pressure;
- diaphragm cross-wall concept;
- ground improvement by deep mixing methods;
- jet-grouted slabs or ribs.

For excavations below the water table in cohesionless or layered soils, the supporting wall will always have to extend some distance below the base of the excavation to prevent a hydraulic uplift or heave type failure. The failure mechanism depends on the specific problem and in particular to what extent there are variations in hydraulic conductivity within the deposit. Potential failure mechanisms can broadly be divided into two categories:

1. A "piping heave" type failure as defined in classical textbooks (e.g. Terzaghi, Peck and Messri, 1995), which generally is the mechanism in fairly homogeneous soils, ref. Figure 36. The failure is caused by upward seepage gradients (or pore pressures) that are so large that they lead to zero effective stresses below the base of the excavation. The failure is usually of a progressive nature, in the sense that it often starts with a local heave and outburst of water close to the wall where the seepage gradients normally are the largest, and then transgresses into a severe piping erosion processes that eventually can involve also the soils on the outside of the wall and create major sinkholes.
2. In strongly layered soils the mechanism can be a more or less uniform lifting of a soil plug from a high-permeable layer (relatively speaking) located some distance below the toe of the wall, Figure 37. This can be looked upon as a question of vertical equilibrium of the "soil plug" from the bottom of the excavation and down the top of the high-permeable layer. In this case, and in particular if the plug consists of clay type soil, the positive effect of interface wall/soil friction may be accounted for. If there are low-permeable cut-off layers below and above the permeable layer, the failure mechanism may be less dramatic than for the case (1). The tendency for uplift and erosion will then often stop because pore pressures will drop hydraulic contact through the plug once is established.

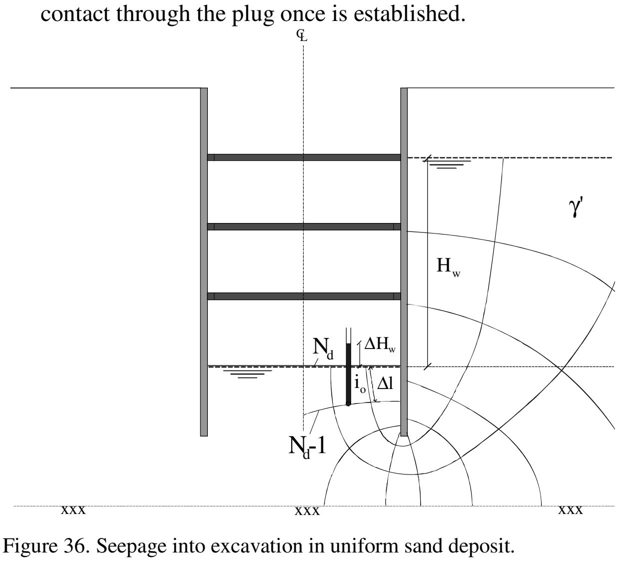


Figure 36. Seepage into excavation in uniform sand deposit.

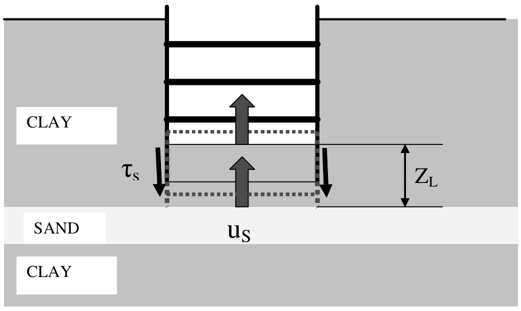


Figure 37. Illustration of hydraulic plug uplift failure from confined sand layer.

Determination of transient and steady state pore pressures under and around the excavation is the key to determine hydraulic uplift stability. The analysis is very sensitive to variations in thickness and relative permeability of the soils below the water table, and in hydrological boundary conditions. Thus thickness and lateral extent of layers with different permeability requires careful mapping.

The following methods or combination of these can be used to improve the hydraulic heave or uplift stability:

– a supporting cut-off wall or grouted wall that is reasonably watertight and extends into sufficiently deep low-permeability layers that can act as an effective horizontal cut-off layer. Note that the required permeability and thickness of a natural horizontal cut-off layer is to be considered on a relative scale compared to the layers above it. For example, the "cut-off layer" could very well be a silty fine sand layer if the layer above is medium to coarse sand.
– a grouted "cut-off plug ". The grout plug must be placed sufficiently deep to satisfy "plug uplift" stability. Such grouting can be a challenging task. The mostly used methods are repeated "tube-de manchette" grouting with chemical grouts with very closely spaced holes, and jet-grouting.
– reducing pore pressures by active pumping-well systems placed in the soil on the outside and/or the inside of the excavation, or by passive relief wells inside the excavation. In both cases it is crucial to pay strict attention to filter criteria to avoid internal erosion of fines, which could lead to substantial settlements and damage to surrounding structures, or to the new structure to be built inside the excavation. Internal erosion could also if it is not stopped eventually lead to a complete blow-in and collapse of the excavation.
– Over-excavating under water and replacing the over-excavated soil by a soil having a significantly larger permeability and at the same good filter properties. The stability against "plug heave" must still be verified.

For an excavation supported by ground anchors, it is necessary to carefully analyze the required free length of the anchors, by documenting the stability of a failure body that extends beyond the anchor zone. The capacity of the soil anchors must in general be verified by proof testing after their installation, and additional anchors installed as required to meet the design requirements. Possible negative effects of interaction between individual anchors must also be considered. This is particularly important for closely spaced ground anchors installed in clays (spacing less than say about 2 meters).

4.1.2 Loads

Loads in struts or anchors and bending moments in the supporting wall are primarily governed by:

– In-situ stresses including pore pressure conditions;
– The strength and stiffness of the soils involved (drained and undrained);
– In particular for soft soils the depth to firm strata or bedrock below the base of the excavation;
– Depth, length and width of the excavation;
– Spacing of struts/anchors;
– Preloading of struts/anchors;
– Stiffness of the supporting wall.

For excavations in soft clays the bottom heave safety factor has a major impact on the loads. This was clearly demonstrated by Karlsrud and Andresen (2008) on basis of parametric FEM analyses for a 10 m deep excavation in clay supported by sheet pile wall and 4 strut levels, see for instance Figure 38.

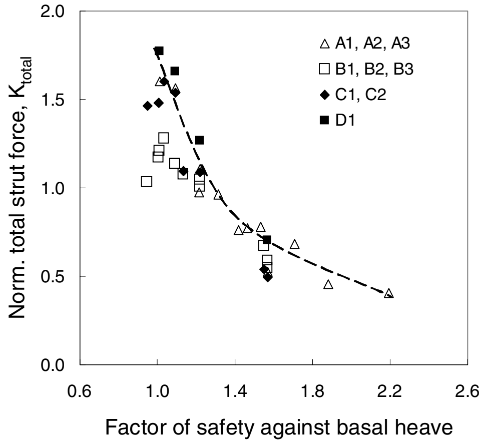


Figure 38. Normalised sum of maximum strut loads, K_{total}, in relation to bottom heave safety factor for excavation in soft normally consolidated clay (after Karlsrud and Andresen, 2008).

Note that the normalized sum of strut loads, K_{total}, in Figure 38 is defined as the sum of maximum strut load at any one level at any stage of excavation divided by the load corresponding to vertical overburden pressure, e.g.

$$K_{total} = \sum P_{max}/0.5\gamma H^2 \quad (4)$$

4.1.3 Deformations

For excavations in soft to medium stiff clays, it is a well established fact that the depth of the excavation and depth to firm bottom have a major impact on expected deformations. Clough et al (1979) and Mana and Clough (1981) showed on basis of FEM analyses and field data that the impact of these two factors are also closely related to the bottom heave safety factor, e.g. Figure 39. This has also been confirmed by the more recent parametric FEM analyses for excavations in soft clays by Karlsrud and Andresen (2008) as referred to above. They used a more realistic non-linear and anisotropic soil model for the clay than the isotropic elasto-plastic model used in the early works of Clough et al (1979). Figure 40 shows that the maximum wall deflection increase very rapidly from about 0.2% of the excavation depth when the apparent bottom heave safety factor, F_{ba}, is greater than about 1.8, to 0.5 % when F_{ba} is about 1.4, and to 2 % when F_{ba} approaches 1.0. This agrees reasonably well with the observed performance summarized by Mana and Clough (1981) in Figure 39.

Note in relation to Figure 40 that the open square symbols that relatively speaking show the smallest displacements are for cases with a wall toed into bedrock 5 to 10 m below the base of the excavation, and without accounting for the effects that it would have on the safety factor (e.g. the safety factor was computed assuming the wall stops at the bottom of the excavation).

Following the early works by Peck (1969), describing and grouping expected ground settlements in connection with supported excavations, a fairly large number of studies have been undertaken to collect and systemize observational data. Some of the more recent studies are briefly reviewed in the following.

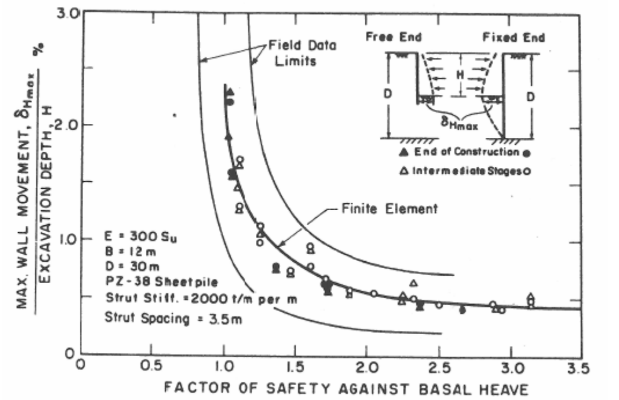


Figure 39. Normalised maximum wall movement against basal heave safety factor, from Mana and Clough (1981).

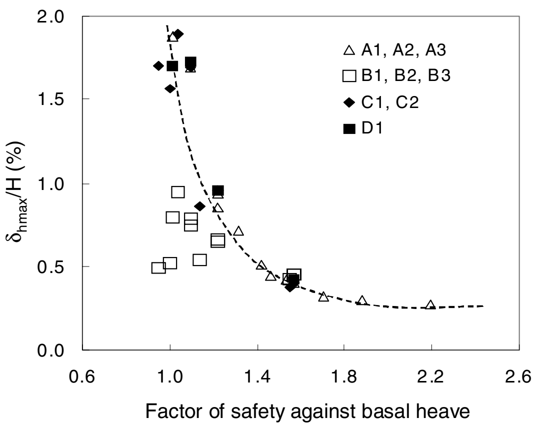


Figure 40. Normalised maximum wall displacement against basal heave safety factor from parametric FEM analyses by Karlsrud & Andresen (2008).

Long (2001) reviewed past work and established an updated database for ground movements associated with deep excavations involving 269 case records with strutted, tie-back anchored, or top-down supported walls, and 27 cases of cantilever walls. In a similar manner as Clough and O'Rourke (1990), Long tried to relate the maximum lateral wall movement and ground settlement to excavation depth, bottom heave stability (when soft soils occur below the base), depth to firm strata and system stiffness. The system stiffness was defined by Clough et al (1989) as:

System stiffness $K = EI/\gamma_w s^4$ (5)

E = Modulus of wall
I = Section modulus of wall
s = Vertical spacing of struts/anchors
γ_w = Unit weight of water

For stiff soils below the base level, Long grouped the observational data in relation to thickness ratio of soft/stiff soils, h/H. For soft soils extending below the base Long related the displacements to the bottom heave safety factor, F_b, as in other studies discussed above.

Figure 41 shows data for cases with mostly stiff soils (h/H<0.6, where h = thickness of soft clay). The data set suggests generally smaller maximum lateral displacements than previously suggested by Clough & O'Rourke (1990), and mostly within a range from 0.05 to 0.3 % of H. Based on Figure 41, Long concluded that system stiffness did not seem to be such a significant factor in controlling displacements as the past work of Clough and co-workers have suggested.

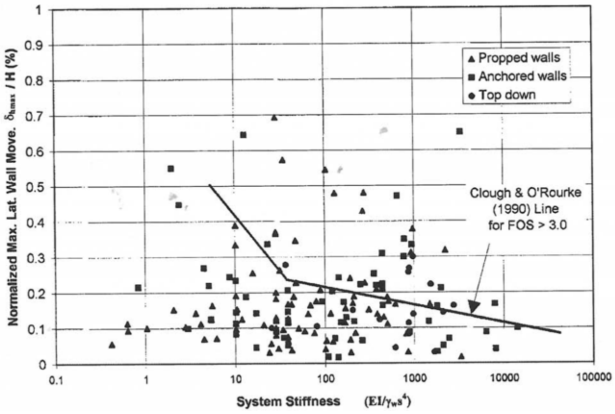


Figure 41. Normalized maximum lateral wall movement versus system stiffness for stiff clay below the base and h/H<0.6 (from Long, 2001).

The same conclusion can be drawn from the data presented by Moorman (2002, 2003, 2004), who extended further on earlier data bases, now including 591 case records, whereof about 530 cases after 1980. As an example, when studying the individual data points in Figure 42 it is hard to see any significant correlation to system stiffness as suggested by the curves from Clough et al (1989) included in the figure.

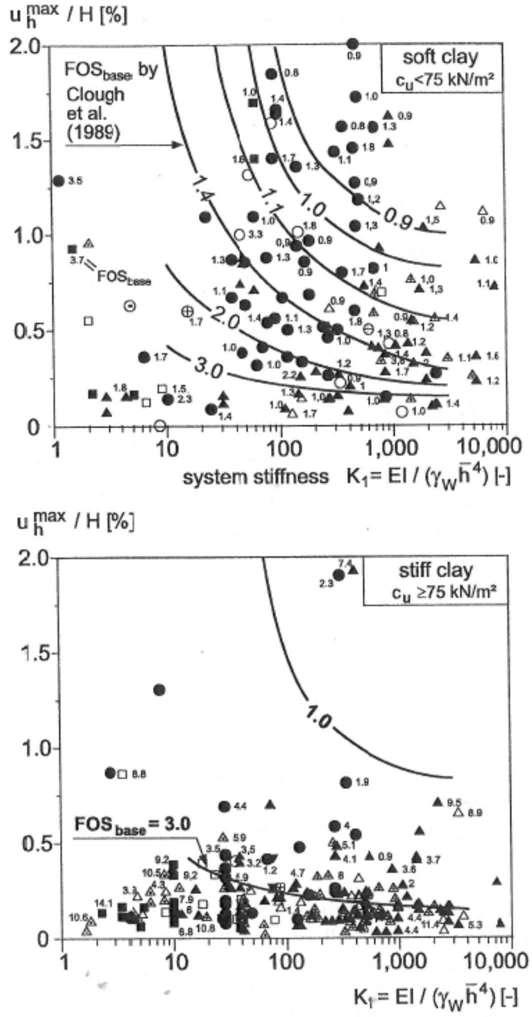


Figure 42. Example of relationship between normalized maximum lateral displacement, u_h^{max}/H, system stiffness and safety factor correlations proposed by Clough et al (1989). From Moorman (2004).

There are a number of reasons for why the system stiffness should not be such a controlling factor for expected displacements as previously suggested. A main point may be that the depth of stress relief below the base of an excavation for a given excavation step is much larger than the strut spacing, and the wall must displace to achieve a new equilibrium state for the imposed stress relief. The depth of such incremental displacement depends primarily on the width of the excavation (e.g. of the unloaded area), stiffness and strength of the soil below the excavation level and on the wall stiffness. The spacing of struts above the excavation level impacts only to a limited extent the rotation stiffness of the wall below the lowest strut, and indirectly the incremental stress relief.

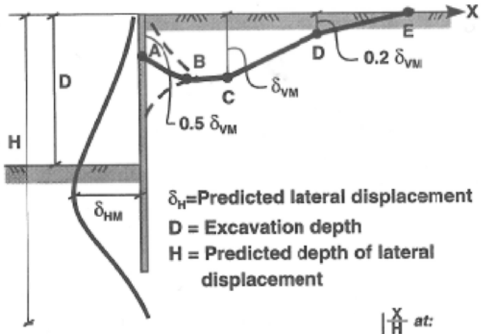
For soft soils extending well below the base the data presented by both Long (2001) and Moorman (2004) confirm the great impact of the bottom heave safety factor on displacements as also demonstrated above.

There are many factors in addition to the bottom heave safety factor that can have a significant impact on displacements, and that are not easily captured by simple correlations. This includes factors such as:

1) Depth of excavation when the first support is installed (initial cantilever stage).
2) Depth of excavation below an anchor or strut prior to its installation.
3) To what extent continuous and good contact is ensured between wall, walers and struts.
4) If or to what extent a sectional excavation and strutting/anchoring procedure is followed. The time element from excavation to strutting/anchoring is in this respect also of importance.
5) Pre-stressing level of struts and anchors
6) Disturbance to the soils below excavation level by the excavating equipment.
7) Disturbance of the soil due to installation of ground anchors
8) Length of soil anchors
9) Pore pressure reduction and consolidation caused by ground water leakage through the wall, up into the base, or thorough anchor holes.
10) Disturbance associated with pile driving inside or outside the excavation.

Moorman (2004) also summarized data on observed settlements of the ground surface, and compared that to previous studies and correlations presented by for instance Peck (1969), Goldberg et al (1976), and Clough and O'Rourke, (1990). New and old data suggest that the maximum ground settlement typically lies in the range 0.5 to 2.0 times the maximum lateral wall displacement, but factors as low as 0.2 have been observed in some cases.

The lateral extent of ground settlement has most commonly been correlated to the excavation depth, e.g. Peck (1969), Moorman (2004) and Wang et al (2005). Karlsrud (1986) suggested however, that the depth from the ground surface to firm bottom is principally a better normalizing parameter than the depth of the excavation or the depth of the wall. Karlsrud (1997a) later proposed to use Figure 43 to determine the settlement profiles from expected lateral displacements. Note that Figure 43 was mainly based on data from sites with soft clays or layered clays and loose to medium dense sand and silts. The dashed lines close to the wall reflects impact of the potential for movements of the tip of the wall. Thus for structures laying at distances from the wall smaller than 0.2 times the depth to zero lateral displacement (x/H<0,2), the settlements can be quite uncertain.



Soil condition	$\frac{\delta_{VM}}{\delta_{HM}}$	$\frac{X}{H}$ at: A	B	C	D	E
Mostly clays	0.7-1.0	0	0.2	0.4	1.0	1.5
Mostly sands	0.5-1.0	0	0.2	0.3	0.7	1.0

Figure 43. Relationship between wall movement and ground settlements as proposed by Karlsrud (1997a) for soft/loose soils.

There are a number of factors that impact the relationship between ground settlement and horizontal displacements that are not easily captured by simple empirical diagrams. This includes the factors 7) to 10) listed above, but in addition comes factors like:

a) The width of the excavation, and thus, the extent of vertical stress relief under and to the sides of the excavation. For wide excavations and soils extending deep below the base of the excavation, the stress relief may actually cause a potential for swelling (heave) type deformations within some distance from the excavation, which partially will compensate for the settlements resulting from wall movements.
b) Distribution of external loads from existing buildings or other structures, including effect of pile foundations of nearby buildings or structures.
c) The actual ground conditions and the detailed stress-strain relationship for the soils involved during both drained and undrained loading.
d) 3D-effects, e.g. location relative to the end walls of an excavation.

Boone and Westland (2005) expanded further on the empirical procedures like those proposed by Clough and O'Rourke (1990) and Boone (2003). The method accounts for system stiffness as defined by eq. (5), preloading level, strut stiffness, strut removal, excavation width and unloading/reloading stiffness of the soil. In addition it accounts for the different impact of concave (bulging) and spandrel (cantilever) type lateral ground movement as originally proposed by Hsieh and Ou (1998).

As regards 3D effects, that issue was addressed by Roboski and Finno (2006). Based on detailed observations of ground movements along excavations in Chicago, Tokyo and Taipei, they proposed a general error function to describe how displacements would deteriorate along an excavation from the central part towards and passed the end walls. Their study suggests that the potential for differential ground movements can be even larger along the ends of an excavation than perpendicular to the excavation, and is therefore an aspect to pay attention to.

Pore pressure reduction and consolidation settlements referred to under point 9) above can be quite an important problem in relation to excavations in soft clays with permeable soil layers or permeable bedrock above or within some meters distance below the base. If such permeable layers are not properly sealed off, the pore pressure reduction can spread out laterally several hundred meters as for instance documented by Karlsrud (1990) and Braaten et al (2004). Such permeable layers will act as under-drainage and start a consolidation process upwards into the soft clay layer. The resulting consolidation settlements can become very significant, easily 5-20 cm and even more in extreme cases. Significant pore pressure reduction and settlements have also been observed due to installation of tie-back anchors or bored piles that intercept permeable layers within or at the bottom of soft clay layers.

The very process of installing ground anchors can also cause disturbance of the surrounding soils as well as “overcutting” effects, both leading to settlements of the ground. The effects and potential for settlements depend on the detailed drilling and grouting procedures applied as well as on workmanship.

4.2. *Measuring Excavation Performance*

It is vital that the scope of a performance monitoring program is defined and understood by all parties involved. Karlsrud (1997b) proposed that the scope could be divided into the following seven functions or purpose categories A) through G), or a combination of these:

- Category A): Verification of the basis for- and soundness of- the design, e.g. that the overall performance of the design and the various elements (wall, struts, anchors, ground improvement) are as anticipated.
- Category B): Detection of unacceptable performance that may lead to failure or have serious consequences, in due time for remedial measures to be undertaken.
- Category C): To help optimize the design, e.g. a deliberate use of a “design as you go”, “observational” or “interactive” design approach.
- Category D): To document the influence and consequences on surrounding structures.
- Category E): To document and verify that the quality of the construction works are as planned.
- Category F): Provide data that may be useful for improving numerical, analytical or empirical design tools or procedures for works of similar type and in similar ground.
- Category G): To provide data that will help enhance general understanding of ground response to construction works, hereunder to help calibrate general soil models and analytical tools.

The extent of and degree of sophistication of the performance monitoring program will vary both in relation to categories A) to G) as defined above, the overall safety level, potential consequences of the excavation on the surroundings, and if or to what extent there are well documented past experiences with similar design in similar ground.

It is beyond the scope of this paper to define or recommend performance monitoring schemes that cover all relevant cases, but Tables 6.a) and b) attempt to weigh the usefulness of the main types of measurements in relation to category. A value of 5 represents high priority- a must, and 1 low priority – not strictly required.

Tables 6.a) and b) are applicable for a design where the overall safety level is deliberately chosen to be fully optimized in relation to costs and with respect to what applicable standards allow. If a conservative design is deliberately chosen, the weighted value set for instrumentation in relation to categories A) to E) will drop.

The extent of instrumentation e.g. number and spacing of different types of measurements will also depend a lot on the variability of site conditions along and normal to the different sides of the excavation.

One of the major developments in practical use of instrumentation systems has in recent years been the possibility of real-time fully automated monitoring of essentially all parameters of interest. Data can be communicated through satellite or cell phone systems and fed into a database system made accessible to all parties that are involved. Measurements can automatically be compared to pre-set alert and alarm levels and those responsible can be automatically called up on cell phone when alert and/or alarm levels are reached. There are few published examples where this has been taken all the way for all types of measurements, but there are good examples of active use of a commonly shared data base system, e.g. Van der Poel et al (2005) and Finno et al (2007).

Table 6.a). Weighted value of different types of deformation measurements (value 5 high-a must, value 1 low-not required).

Category	Hor. displ. at wall	Hor disp. behind wall	Ground surface settlement	Vertical distribution of ground movements	Settlement of surrounding structures	Tilt and strain in surrounding structures
A-Verify basis for design	5	3	4	1	3	1
B- Warning against failure	5	3	2	1	2	1
C- Observational design approach	5	3	4	1	4	2
D- Influence on surroundings	4	3	5	2	5	4
E- Verify quality of construction	4	2	4	1	3	2
F- Improve design rules	5	4	4	2	4	3
G-Enhance knowledge	5	5	5	5	4	3

Table 6.b) Weighted value of other measurements (value 5 high-a must, value 1 low-not required).

‘	Loads in struts or anchors	Temperature in struts	Strain in wall	Pore pressure within the excavation	Pore pressure outside the excavation	Earth and pore pressures against the wall
A-Verify basis for design	3	3	3	1-5	3-5	1
B- Warning against failure	5	4	3	1-5	1-3	1
C- Observational design approach	5	3	4	1-5	1-3	1
D- Influence on surroundings	1	1	1	1-2	3-5	1
E- Verify quality of construction	4	2	1	1-4	1-4	1
F- Improve design rules	5	3	3	1-4	1-4	4
G-Enhance knowledge	5	3	5	2-4	3-5	5

Some additional comments are given in the following to some specific types of measurements, in particular to the more recent developments of instrumentation systems that represents potential for improvements compared to the past.

a) Horizontal displacements (lateral wall or ground movements)
   Installation of inclinometer tubes in the ground is vital to get a good picture of lateral displacements in the ground. There has been a significant improvement in the accuracy of the inclinometer torpedos commercially available. In the authors experience absolute displacements over a 20 m depth can be measured with an accuracy of about 0.2 to 0.5 mm. This will normally require that readings are taken for every 0.5 m in two directions (torpedo is run in one direction first, then turned 180^0 and run in the other direction)
   Use of fixed inclinometer strings mounted in holes gives the possibility for fully automated measurements, and has been used with success on many projects. The accuracy will in this case depend on the spacing of inclinometers in the string, which of course also is a cost issue.
   A more recent and interesting development is the possible use of fiberoptic techniques. By installing two cables diametrically in a hole the distribution of angular distortions and thus, the lateral displacements can be obtained. To the authors knowledge this possible application has not as yet been fully tested out.
b) Ground surface settlement
   The common way at getting at this is by installing robust surveying objects at or just below the ground surface. A surveying pin or prism is of course needed at the top of the object. The major challenge here lies in the potential for damage of the objects to be leveled, and their interference with construction at the site. Readings can in principal be fully automated by use of precision leveling total station equipment fixed at some place in the vicinity where no deformations are expected.
   To avoid interference with traffic and construction, settlement hoses or tubes can be installed in the ground, through which a gage for hydraulic pressure measurements is pulled along. The change in hydraulic pressure relative to initial readings, and a known settlement at the end point (obtained by surveying), gives the settlement profile along the hose.
   The accuracy of the system is in principal reasonably good, but in practice results have in the authors experience been more uncertain. In principal it should be possible to use inclinometer tubes, fixed inclinometer strings or fibreoptic cables for the same purpose, but to the authors knowledge that has never been done so far.
c) Distribution of vertical ground movement
   There is a variety of extensometer system on the market which can give results of very high accuracy. By use of LVDT displacement transducers or similar it is also possible to fully automated readings, but deformations at ground level are needed as a reference.
d) Deformations of surrounding structures
   Fully automated high precision total stations with measuring prisms mounted on buildings are now commonly applied on larger projects. It seems to be gradually replacing ordinary surveying, and is specially worth considering when frequent measurements are needed over a long time period. The accuracy of absolute displacements relative to assumed fixed point seems to be of the order 0.5 mm or even less.
   As discussed in the subsequent section 4.3, the lateral strain in buildings is an important component contributing to damage. It can be directly measured by extensometers, that if needed, can be fully automated.
   A more recent and potentially even more interesting development is the use of remote sensing techniques. For mapping larger areas, the differential SAR Interferometry (DInSAR) technique is now well developed and used for detecting surface changes. It uses radar images generated from orbitted satelites. Surface or building settlements are established by comparing multiple radar images. The technique first became well known after an image of the Landers Earthquake deformation field was published in the journal Nature in 1993 (Massonnett et al., 1993). The method has the potential to detect millimetric surface deformation along the sensor – target line-of-sight. Numerous studies of urban subsidence using radar interferometry have been published (Amelung et al., 1999; Fruneau and Sarti, 2000; Galloway et al., 1998). Both linear trends and seasonal fluctuations can be identified (Colesanti et al., 2003a; Colesanti et al., 2003b). The accuracy may be about 2-3 mm. The technique requires object with fairly sharp contours and geometric changes. It is therefore particularily well suited for buildings, but less suited for plain ground.
   On a more local level, ground based equipment for radar or laser light array scanning (LiDAR) are now commercially available. The LiDAR technology was originally developed for the air and auto industries. Van Gosliga et al (2006) and Lemy et al (2006) have used the technique to measure tunnel wall displacements. There is no question that it can also be used to determine deformations of buildings affected by a tunnel or excavation. The resolution of such measurements is becoming very impressive, and changes in deformations in all planes can presently be detected with an accuracy of 1 to 3 mm depending of what system is used.
e) Loads and pressures
   There are to the authors' knowledge no really new and major developments in relation to measuring loads in struts and anchors, pore pressures or earth pressures in the ground or against walls. Most systems are based on vibrating wire or resistance type strain gages. The accuracy of the sensors may vary depending on the manufacturer, and the choice of sensor system will depend on needs for long term stability, and sensitivity of sensor systems to factors like absolute and differential temperature, electrical noise etc.

4.3. *Acceptable deformations*

The potential for building damage caused by settlements were early on related to rather simple angular distortion or differential settlement criteria, and the type of building structure under consideration (e.g. Skempton, and MacDonald, 1956; Bjerrum, 1963). It has later been recognized that a more complete representation of a buildings deformation pattern is required to address the damage potential. Burland and Wroth (1975) introduced a set of displacement criteria as illustrated in Figure 44, and clearly differentiated between sagging or hogging type displacement patterns when it comes to damage potential. They also discussed and assessed the damage potential based on idealized beam theory accounting for building stiffness in a simplified way, and stressed the importance of the potential for horizontal and diagonal tensile strains as a main cause for building damage. Notwithstanding such strains, the absolute settlement and absolute angular distortion are by themselves important parameters, as they can be visually observed or felt by the building owners. As an example, if an excavation causes a uniform tilt of a building of say 1:300 without any angular distortions and no cracking or building damage as a result, the property owner would still, and rightfully so, consider this as damage due to "esthetic" loss and the inconvenience of having tilted floors and walls.

Table 7 defines the settlement parameters defined in Figure 44 and examples of acceptance criteria that were used in relation to design of parts of the Taipei Metro in the late 1980's in which the second author was involved.

Table 7. Example of settlement criteria used for excavations in connection with Taipei Metro.

Building type	Settlement δ_{max} (mm)	Absolute rotation θ_{max} (rad)	Angular distortion β_{max} (rad)	Hogging ratio Δ/L (rad)	Sagging ratio Δ/L (rad)
Multi-storey framed building on raft foundation	45	2x10^{-3}	2x10^{-3}	0.8x10^{-3}	1.2x10^{-3}
Concrete framed building on footings	40	2x10^{-3}	2x10^{-3}	0.6x10^{-3}	0.8x10^{-3}
Brick building on footings	25	2x10^{-3}	0.4x10^{-3}	0.2x10^{-3}	0.4x10^{-3}
Temporary structures	40	2x10^{-3}	2x10^{-3}	0.8x10^{-3-}	1.2x10^{-3}

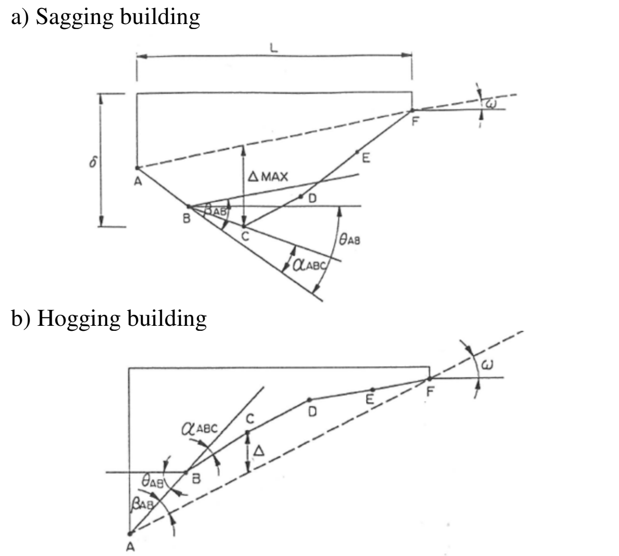


Figure 44. Settlement parameters used to address damage criteria.

Boscardin and Cording (1989) expands further on use of the limiting tensile strain, ε_{lim}, introduced by Burland et al (1977) as a damage criteria and proposed the damage criteria presented in Table 8. Burland (1997) later proposed also to include the hogging ratio with the limiting tensile strain criteria as damage criteria, Figure 45.

Son and Cording (2005) presented results of numerical analyses of buildings, and model testing carried out to develop a further understanding of the damage potential of buildings. They combined the results with actually observed building damage, Figure 46.

Table 8. Damage categories proposed by Boscardin and Cording (1989).

Damage category	Normal degree of severity	Limiting tensile strain, ε_{lim} (%)
0	Negligible	0-0.5
1	Very slight	0.05-0.075
2	Slight	0.075-0.15
3	Moderate	0.15-0.3
4 to 5	Severe to very severe	>0.3

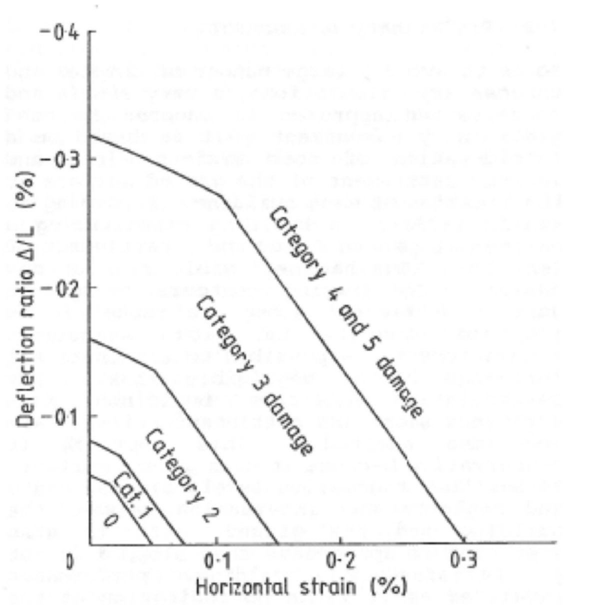


Figure 45. Relationship of damage category to deflection ratio and horizontal tensile strain for hogging (L/H = 1), after Burland (1997).

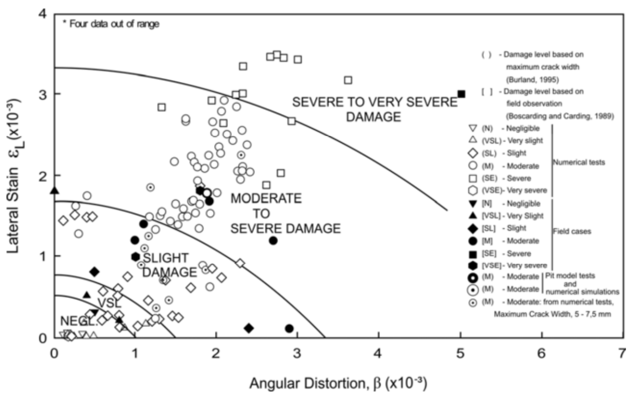


Figure 46. Damage categories proposed by Son and Cording (2005).

Son and Cording (2005) actually recommended a stepwise damage assessment. The last step, involving a detailed soil-structure analysis, is definitively feasible with present FEM capabilities. Not only 2D but also fully coupled 3D analyses are now possible and becoming more common in practical use, e.g. Schwab et al (2007).

It is however, a challenge in such modeling to capture realistically all factors contributing to ground movements and how they may vary in space. It may require some fudging with input soil stiffness parameters and their spatial variation to ensure that predicted ground movements are realistic in relation to past experiences under similar ground and excavation conditions.

Finno et al (2005) have proposed a simplified model for representing building systems based on a "laminated beam" concept, which represents an improvement compared to the simple "deep beam" theory which was used by Burland and Wroth (1975) and Burland (1997). This concept transfers the complete stiffness of multi-storied buildings into that of a laminated beam, accounting for the impact of stiffness of different parts of a building, e.g. floors, walls, partitions etc. The theory was based on the work by Voss (2003). Finno et al (2005) show by a real example, that the laminated beam theory gives better prediction of tensile cracks as observed in a school building due to an excavation, than the simpler deep beam approach.

4.4. *Prediction and Evaluation of Excavation Behavior.*

Empirical methods exist for predicting both loads in struts and anchors and ground movements, as discussed in section 4.1. In cases where there are important structures or infrastructure in close proximity of the excavation, and even though the margin of safety is large against any kind of failure mechanism (say FOS larger than 3.0), empirical methods should primarily be used as guidelines, and supported by or replaced by use of more refined analyses.

With use of modern FEM analyses and up-to date site investigations to define stress-strain and strength parameters of the ground, it is possible to make good predictions of loads and expected lateral and vertical displacements. One important aspect in this connection, as also emphasized by Boone and Westland (2005), is that the soil model must properly account for the strong non-linearity of the stress-strain behavior of soils, and in particular the much higher soil stiffness in unloading/reloading than for first time loading. Failure to do so will have particularly large impact for predicted vertical ground movements. It is still recommended to compare such FEM analyses against the most relevant past case records, and in particular carefully consider special factors that may impact the magnitude and pattern of deformations as well as loads.

Parametric studies are also generally recommended to get a reasonable picture of upper and lower bounds to predicted loads and displacements.

As observed by Karlsrud and Andresen (2008), it is unfortunate that many important projects are designed on basis of rather rudimentary soil investigations. It is really a challenge to our profession to enhance the use modern up- to- date procedures when it comes to determination of the true in-situ stress-strain and strength characteristics of the soils we are dealing with in our designs. Presently, it seems like that numerical capabilities are far more advanced and incorporated in design practice than the determination of relevant soil parameters.

Many designers still use fairly simpler approaches like beam-on-spring type methods for design. Such models require careful consideration of what the appropriate equivalent spring stiffness should be, and it is very important that the model captures the limiting active and passive pressure at every step of an excavation. "Default" spring stiffness values suggested in some of these programs often fail to capture the many factors that actually impact the equivalent spring stiffness, e.g. Karlsrud (1997a). If the limiting pressures and spring stiffness are appropriately assessed, the analyses can give reasonable estimates of loads and lateral displacements. Equivalent spring models are otherwise best suited for walls extending to, or toed into, a firm base located within a depth of about 0.5 H of the base of the excavation. When using such equivalent spring models, it is necessary to use empirical correlations to get at the ground settlement profile, e.g. Figure 43.

4.5. *Dealing with observed performance.*

A main reason for instrumentation and performance monitoring is to ensure that the excavation is safe from collapse. Measurements are by themselves of little value unless realistic acceptance criteria in terms of alert and alarm levels have been set prior to construction.

The type and magnitude of alert and alarm levels to be set depend on the two main design criteria:

a) Ensuring that the safety level of the excavation is acceptable, e.g the margin of safety against a complete collapse of the excavation is at all times satisfactory. This includes both the issue of overall stability of the excavation and the potential for overstressing of the wall or the strutting/anchoring system.
b) Ensuring that displacements are within acceptable limits in relation to the potential for causing damage to neighbouring structures or utilities.

If an alarm level is reached that suggests that the safety of the excavation is at stake, it is vital that appropriate remedial actions have been planned for and can be rapidly implemented as needed. Implementing remedial actions may be easier said than done, considering also that the time element is important, and that safety of workers must be ensured also during implementation of such remedial measures.

In most cases the first and primary element of remedial action would be to backfill the excavation with any kind of material readily available, including soil, water or structural materials like steel or concrete blocks. To excavate and unload the ground on the retaining side is also a measure to be considered as short term remedial measures

Additional strutting, ground improvement or other reinforcing measures would normally be carried out as a second step, subject to a critical review and understanding of the cause of the critical condition that has arisen.

To make a decision to immediately implement such remedial actions, more or less on the spur of the moment, is a major challenge to the parties involved, and requires:

- a clear definition of responsibilities and chain of command between the parties involved (the owner, designer and contractor).
- at least some of the decision maker(s) must have a solid geotechnical engineering background, and especially with design and construction of excavations.
- the decision maker(s) must be available on call at all times within say maximum one hour.
- a data collection and presentation system which ensures that monitored data are input and processed within a few hours after measurements have been made, and that the results can be readily compared to alert and alarm levels set. If the design in the outset is made with small safety margins and the consequences of a failure is large, on-line monitoring should be considered for critical elements.

To set alert and alarm levels in relation to measured loads in struts or anchors is generally an easier task than for displacements, as the capacity of structural members is well defined. It is however generally recommended to carry out design analyses that try to capture the sensitivity of the loads to input strength and stiffness of the soils involved, and to have a clear picture of expected load changes when excavating from one level to the next.

As demonstrated in section 4.1, there is a good correlation between maximum displacements and the bottom heave safety factor, but displacements are more difficult to relate to the potential collapse of struts and anchors.

If loads or displacements at any stage of the excavation differ significantly from what is predicted, supplementary analyses should be carried out to find an explanation for the difference. It is therefore of great importance that the designers of the excavation are involved and available during the construction phase.

4.6. *Interactive Excavation Design*

Interactive design is not a well suited approach for deep excavation projects unless it is a long excavation in rather uniform conditions. The reason is that the selected type of excavation support, including wall, struts or anchors, and ground improvement, are not readily exchangeable during the course of the works. This makes it very different from for instance tunneling in rock or other competent ground, where the type and extent of support is more readily adapted to variations in local conditions.

In connection with excavations, the elements that are best suited for interactive design are:

– the vertical and horizontal spacing of soil anchors for anchored walls. The spacing can be adjusted on basis of proof loading of individual anchors.
– the extent of deep ground improvement if and when that forms part of a design, and which may be adjusted after sampling or in-situ methods have revealed what properties have actually been achieved.
– type and spacing of wells to control pore pressures under the base, which can be adjusted based on measured pore pressures.

For long cut-and cover excavations, say longer than about 500 m, one may still consider to vary other elements in an interactive process. It would then be preferable to start off with a conservative design, and then move in a less conservative direction provided that the excavation support performs better than the conservative scenario suggested.

4.7. *Case Histories*

4.7.1 *Learning from failures*

It is an unfortunate fact that failures have been an important source for enhancing our understanding of the real behaviour of soils and the sometimes complex interaction between soil and structures in the ground. It is however, a rather costly way of learning. We should as a profession spend more time on back analyses of well documented cases, parametric studies with advanced numerical tools and soil models, and maybe model testing.

A number of failures or incipient failures causing unexpected and large ground movements in connection with deep excavations have been published over the years. Shirlaw et al (2005) reviewed for instance failures or near failures in connection with deep excavations in Singapore.

A case from which many lessons may be learned is that of the large and dramatic MRT excavation collapse in Singapore on 20 April 2004. The case was subject to investigations by various experts and expert groups set down by the different parties involved, see for instance the Committee of Inquiry (2005), Hieng (2004), Davies (2004), Ove Arup & Partners (2004) and Endicott (2004), and was also briefly reviewed by Karlsrud and Andresen (2008).

Figure 47 shows a typical cross section of the excavation. It was to be 33.7 m deep with a width ranging from about 14 to 21 m. The ground conditions consist of an upper layer of reclaimed fill followed by soft marine clay extending to near the bottom of the excavation. Below the marine clay there is an Old Alluvium (OA) consisting of layered firm clayey silts and silty sands. The wall was to be braced at 10 levels and had two levels of jet-grouted slabs, the lowest one just beneath the base. The wall toed 5 to 10 m into the OA. The failure occurred after excavation below the 9^{th} strut level and the beginning of removal of the upper jet-grouted slab. The complete and dramatic failure developed rapidly and involved a 220 m length of the excavation. It is not the intention herein to present or discuss in any detail cause(s) of the collapse, but rather focus on why the collapse occurred in spite of a rather extensive monitoring program.

Very briefly, the direct cause of the failure was most likely one or more of the following factors:

1) Erroneous soil design parameters. The design analyses were made using the FEM program PLAXIS (2001). In the analyses the designers erroneously chose to use the *undrained effective stress* Mohr-Coulomb (MC) model available in PLAXIS to model the soft marine clay layer. The implication is that the undrained strength used is about 60 % larger than the actual undrained shear strength of this normally consolidated marine clay deposit. Consequently, deformations, loads and safety level were underestimated.
2) Deeper soft clay layer. In the failed area the depth to the OA layer was, locally along the south wall, up to 5 m larger than assumed in the design. This will increase both loads and displacements compared to the design analyses.
3) Under design of struts/walers and connections. The strut/waler connections have in retrospect been shown to have a capacity maybe down to 50 % of the capacity of the struts themselves. The significant rotation of the walls, walers and struts at the connections further reduced their capacity. Such effects of rotation were not accounted for in the original design or the different design reviews made after the much larger than expected wall movements were first observed.
4) The installed capacity of the jet-grouted slabs was probably exceeded. Evidence of this is that the observed wall movements exceeded the typical strain at failure of jet-grouted clay. It is also possible that there were local lack in contact or overlap between each jet-grouted "column".
5) There was a local 3 m wide slot in the diaphragm wall (to make room for utilities crossing the excavation). This slot was in-filled partly by with jet-grouting and partly by steel. It could still have represented a weakening of the wall, but most investigators believe the effect has not been very significant.
6) It has been suggested that prior to the failure there were indications of vertical upward deformations in at least some parts of the excavation, including the king-posts. Vertical movement of the king posts will cause eccentric loading of the struts which will reduce their capacity significantly.

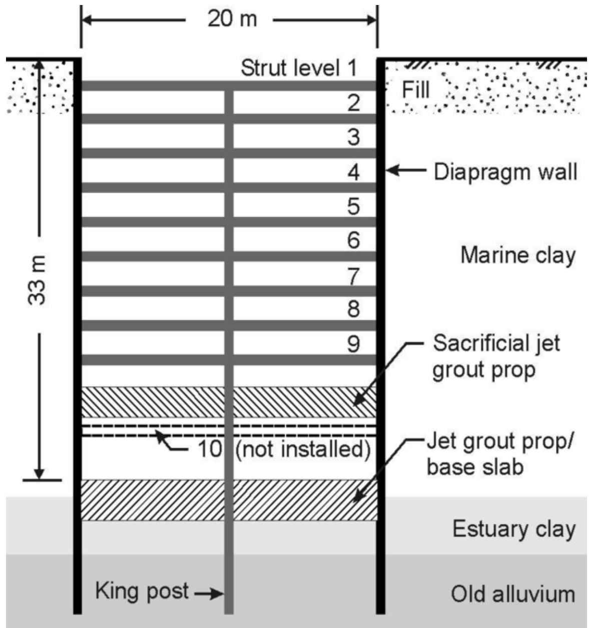


Figure 47. Typical cross section of the MRT excavation in Singapore that collapsed 20 April 2004.

This MRT excavation in Singapore was in the outset the by far deepest and most challenging excavation in Singapore soft clays to that date. Both the owner, and design-and-build contractor was fully aware of that fact. It was therefore planned and implemented a fairly comprehensive instrumentation and monitoring program. Trigger levels were also established, and routines for follow-up meetings established. The trigger level was generally set at 70% of the design level. There were weekly instrumentation review meetings, and special review meetings if the trigger levels were exceeded. If the data suggested that the design levels could be exceeded during subsequent excavation, the contractor should reassess the design and implement remedial measures as seen required.

In the area which failed there were inclinometers at or just behind the wall at both sides of the excavation, settlement markers on the ground surface, and the load in every strut was measured with load cells or strain gauges. There were also 2 piezometers on the outside of the excavation and 1 on the inside, but the latter was damaged and not replaced during excavation.

Figure 48, copied from Ove Arup & Partners (2004), shows the horizontal displacements measured on the north and south side of the excavation after installation of the 2nd to 9th strut level. Note that the 4 last curves for the north side and the 5 last on the south side are displacements measured during excavation for the 10th level struts, including removal of the upper jet grouted slab, in the period 10 to 20 April 2004 when failure eventually occurred. Just before failure, the maximum displacement on the south side had reached about 435 mm and about 200 mm on the north side. The reason for the difference in displacements between the two sides is mainly due to the fact that the depth to the firm old alluvium (OA) layer in reality was about 5 m larger on the south side than on the north side of the excavation, an aspect which was not accounted for in the design. The real depth to the OA is apparent from the inclinometer measurements as the level at which there is essentially zero wall displacement.

Figure 49, copied from Hieng (2004), shows the evolution of maximum horizontal displacement for the south wall. It appears that the displacements had tended to stabilize after the 9 th level struts were installed, but they then accelerated rapidly during the excavation for the 10th level.

In the following it is briefly summarized how the observed displacements were dealt with at different stages of the construction, as documented in the Committee of Inquiry, (2005) report:

1) On 27 February 2004, after excavation to the 6th strut level, the maximum wall displacement, δ_{hmax}, exceeded the final design level of 159 mm. The contractor then produced a 1st revision of the design analyses that fitted the measured displacement. This was primarily done by modifying the stiffness values of the jet-grouted slabs. This gave a new predicted δ_{hmax} of 251 mm at the final stage. This revised analysis was approved by the client and excavation was allowed to proceed.
2) On 18 March 2004, after excavation to the 8th strut level, δ_{hmax} had reached about 280 mm and exceeded the 1st revised δ_{hmax} value of 251mm. On 30 March the contractor presented a 2nd revision of his design analyses, again by primarily adjusting stiffness values. This gave δ_{hmax} of 359 mm, and was also approved by the client.
3) In a letter from the client to the contractor on 15 April 2004, the client expressed concerns because the displacements suggested that the diaphragm wall then had reached its theoretical ultimate bending capacity. The contractor responded that he believed the steel in the reinforcement had reserve capacity (460 MPa in design, 500 MPa according to certificates), and that he (mysteriously) believed that the inclinometer, which was installed just on the outside of the wall, exaggerated the actual radius of curvature by 15 %. Thus, the work proceeded assuming there were still margins of safety left.

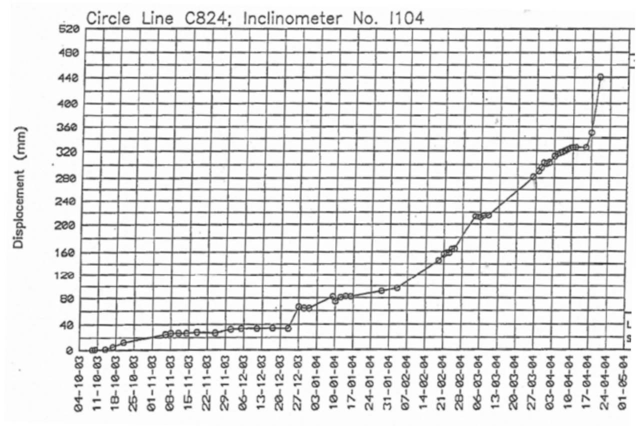


Figure 49. Measured evolution of maximum wall displacement at south side (from Hieng, 2004).

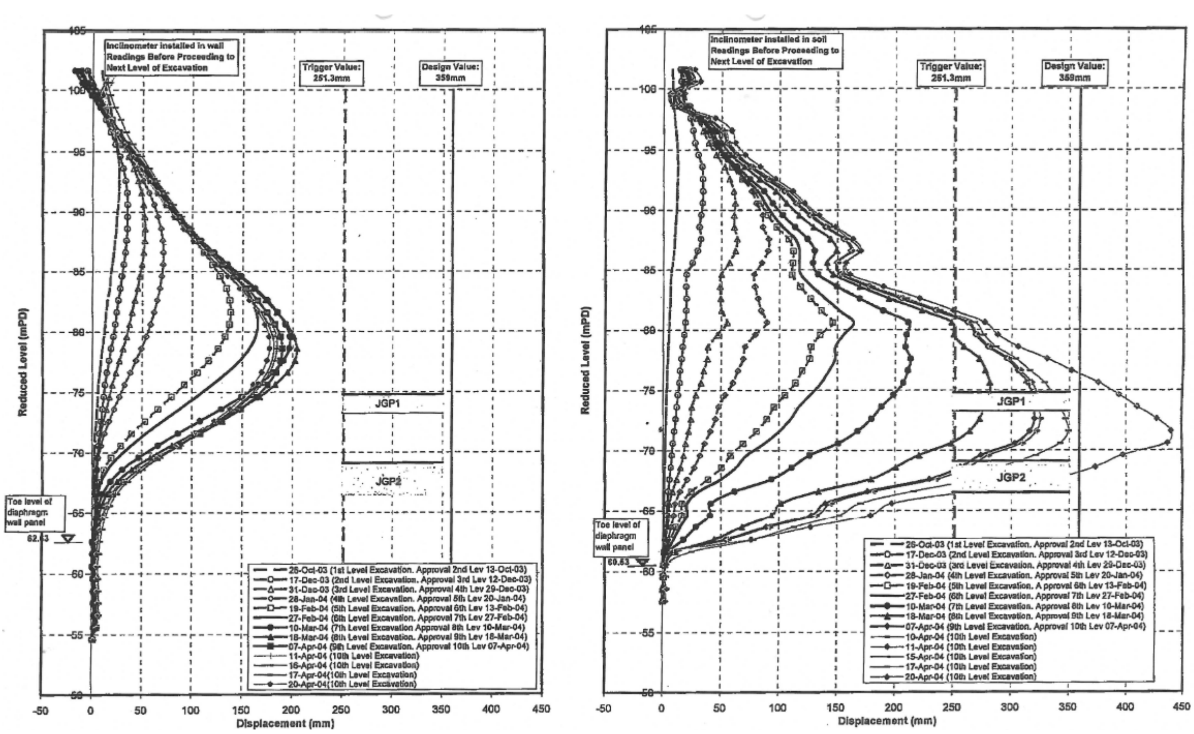


Figure 48. Measured wall deflections, left north side, right south side (from Arup, 2004)

4) On 17 April 2004 the 2nd revised δ_{hmax} value of 359 mm was also exceeded, but does not seem to have lead to any specific reactions.

Figure 50 shows the measured development in load in the 3 lowest strut levels 7 to 9 on 20 April during the last 15 hours leading up to the complete collapse of the excavation. It appears that the loads were stable up until around 10AM. Then the load in the 9 level struts suddenly started to decrease and the 8th level correspondingly increase. The explanation for these load changes was that the strut/waler connection at the 9th level strut started to fail, thereby transferring load to the upper 8th level. The 8th level then eventually also got overloaded, and failed around 3PM, leading to a rapid and progressive collapse of the entire excavation.

That the failure initiated at the 9th level waler/strut connection has been confirmed. At 10 AM workers heard "sounds" from these connections, and visual inspection by an engineer at 12AM revealed that a buckling failure had occurred in the c-type stiffeners used for the waler at the 9th level.

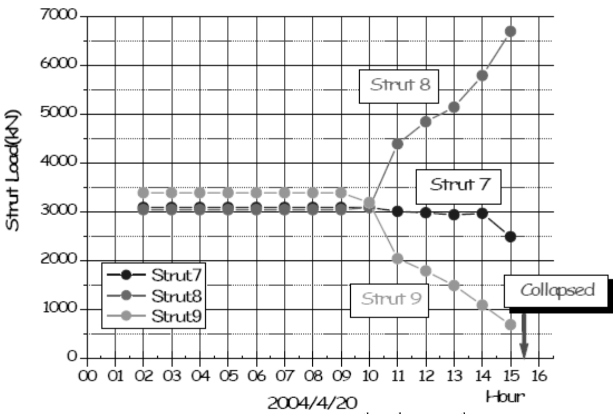


Figure 50. Development in strut loads at 7th, 8th and 9th strut level in the hours before failure occurred (copied from Iwasaki, 2008).

A very important observation that can be made in relation to Figure 50 is that the loads were quite stable up until 5 hours before the failure, and thus, did on their own indicate that failure was incipient. This underscores a statement made by Karlsrud (1997b) in relation to our possibility of detecting the potential for a failure at an early stage of its development:

"One may therefore question whether we fully appreciate the "brittleness" of the overall system, and that rapid and "brittle" failures may occur, not only as a result of the strain softening nature of soils or rock masses involved, but also due to the "brittleness" of the support system…"

In the current case this raises another interesting question: If the loads had been monitored in real-time, with appropriately set alarm levels, responsible people available and called automatically on cell-phone if the alarm level was reached, could then this failure have been avoided? The answer to this is not an obvious "yes". The "lead time" was in practice about 4 hours, and it would be a very challenging task to mobilise for effective backfilling of the excavation or other stabilising measures in such a short time period, considering it also could have happened at night

Thus, a very important lesson to learn from this case is to *at all times* ensure that there is a *real margin of safety* in the construction, and which in general should be in accordance with accepted standards. Actions to improve the overall safety level should be taken when deviation from expected behaviour starts to occur. As a minimum, then at least stop the works and make in-depth checks of the design and observational data, preferably by independent and acknowledged experts.

One can really question why those in charge of the works in Singapore accepted such large deviations between originally predicted wall deflections and what actually was measured. They seem to have been caught in "wishful thinking", hoping for the best, and not willing to accept the burden of admitting that something was fundamentally wrong with the design. This observation may also be applicable to several other failures or cases of large and excessive deformations in connection with deep excavations. Lack of properly qualified and experienced staff on site who closely follows the construction and monitored data, may be another and indirect common cause of failure.

4.7.2 *Learning from performance data*

There is a more or less a continuous flow of information regarding monitored data from new and challenging excavation projects. The relatively large scatter in the data bases for wall and ground movements reviewed in section 4.1 probably comes partly from a lack of consistent and high-quality soils data for the cases involved. In addition there are a number of other factors that are not readily accounted for which causes scatter, such as failure to account for non-linearity of the soils response, impact of the detailed construction procedure actually followed, and impact of ground water leakage into the excavation.

Even for the majority of cases published since 2000 it seems like soil´s data on which design and analyses were based, often come from rather simple and rudimentary soils investigations. Oedometer tests, triaxial or direct simple shear test results on *high-quality* soil samples are scarcely reported. This is also a challenge the profession has to face up to. To make realistic predictions and really learn from back-analyses requires soils data of high quality! As was suggested by Table 6, only knowing lateral wall displacements or ground surface settlements is far from sufficient when we are really going to verify or calibrate numerical models and input parameters.

It is actually surprising how few well documented and back-analysed case records that have been published. More efforts in that direction are also strongly encouraged.

5 TUNNELS

5.1 *Soil Responses to Tunnelling.*

The idealized stress and strain changes around a deep tunnel in soil with negligible cohesion under hydrostatic in situ stress and plane strain condition was described by Negro and Eisenstein (1991). It is reproduced in Figure 51, assuming a fully drained condition and considering that the soil stiffness depends on the mean normal stress and on the degree of shear mobilization. It is assumed also that no reduction in the friction angle is observed upon failure. Excavation is mimicked by the radial stress reduction. From A to B (Figure 51.a), a nearly elastic response is noted, at B some yield develops, at C maximum shear stress peaks and failure is attained soon after, with the mean normal stress reducing faster than the increase in shear. To comply with the failure criterion, the maximum shear stress starts to drop after C (Figure 51.b) and the stress obliquity remains constant (Figure 51.c). In the nearly elastic portion AB, close to zero volume changes takes place as pure shear dominates. Depending on the soil stress history and on the stress level, a small decrease in volume may be observed (Figure 51.d), followed by dilation associated to the faster rate of mean normal stress reduction. The integrations of the radial strains combined with radial stress (Figure 51.e) lead to what is called *ground reaction curve* or *convergence curve.*

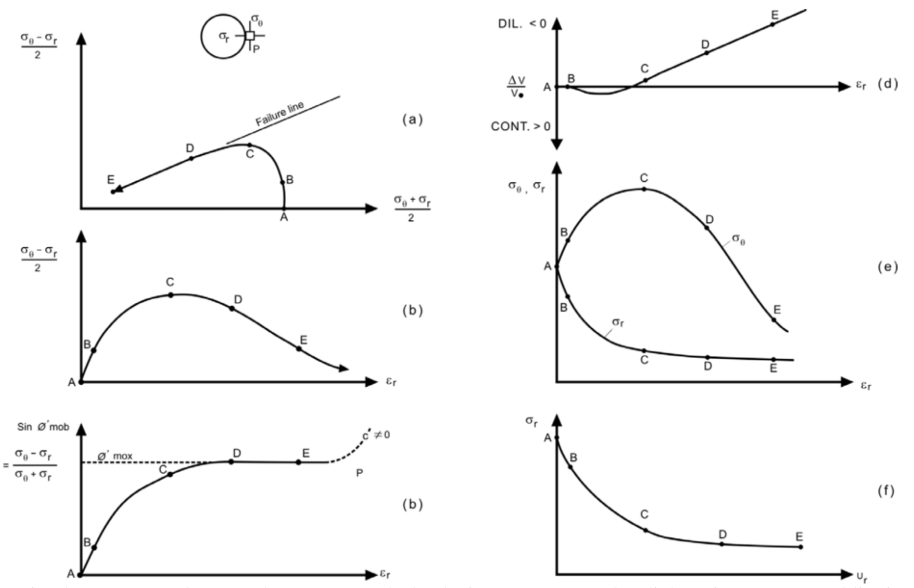


Figure 51. Stress and strain changes around a tunnel: a- stress path; b- deviator stresses and radial strains; c- stress obliquity and radial strains; d- volumetric strain changes and radial strains; e- tangential and radial stresses with radial strains; f- ground reaction curve (modified from Negro and Eisenstein, 1991).

While between points C and D localized failure develops, tunnel collapse will take place between D and E (Figure 52.a) and first yield is noted at B, well before local failure is achieved. Good tunnelling practice in urban scenarios should lead to stress changes confined to stress region AC or AD, in which some yield and plastic straining develops and soil response is non linear.

Atkinson (2000) reviewed the importance of non linear soil stiffness in geotechnical engineering practice and expressed non linear behaviour in terms of *rigidity* (the ratio initial stiffness to failure strength or $E_0/q_f=1/\varepsilon_r$, where ε_r is a reference strain) and of *degree of non linearity* (the ratio failure strain to the reference strain or $\varepsilon_f/\varepsilon_r$). Atkinson (op. cit.) furnished typical values of both parameters for some civil engineering materials, reproduced in Table 9. It is noted that rigidities of soils are greater than that of concrete and steel simply because soils are relatively weaker than other materials. Moreover, the rigidity of softer soils is, surprisingly, larger than that of stiff soils. On the other hand, the degree of non linearity of soft soils is comparable to that of steel and that of stiff soils to concrete.

Table 9. Rigidity and degree of non linearity of some civil engineering materials (partly extracted from Atkinson, 2000).

Material	Initial Stiffness E_0 (MPa)	Failure Strength q_f or $2c_u$(MPa)	Rigidity (E_0/q_f) $= (1/\varepsilon_r)$	Failure Strain ε_f(%)	Degree of non-linearity $\varepsilon_f/\varepsilon_r$
Soft soil	100	0.05	2.000	10	200
Stiff soil	300	0.3	1.000	1	10
Concrete	28.000	40	700	0.35	2
Mild steel	210.000	430	500	30	150

Mitchell and Soga (2005) reviewed soil stiffness degradation of non linear soil behaviour and reproduced a curve representing a stiff clay such as London Clay, with strain levels involved in some geotechnical structures (Mair, 1993) shown in Figure 52. Strain levels related to tunnels where here liberally updated to include modern EPB and slurry technologies, that can reduce strain levels substantially and to include lower quality traditional mining methods, still present in routine practice of some countries. Mitchell and Soga (op.cit.) separated the stiffness degradation curve into four zones, also shown in Figure 52 with limiting strain values that closely correspond to points A, to E indicated in Figure 51: from A to B the *nearly elastic zone*, which Atkinson (2000) refer to as the *very small strain range*, from B to C the *non linear elastic zone,* from C to D the *pre-yield plastic zone* (Atkinson, op.cit., called the range between points B and D as the *small strain* range) and from D to E and beyond, the *full plastic zone* (or the *large strain range,* according to Atkinson, op. cit.). Figure 52 shows also the typical strain levels that can be measured by current laboratory tests (Atkinson, op. cit.).

Ground stress changes by tunnelling induces volumetric and shear straining in the soil, that in prototypes can be inferred by conventional monitoring, including combined slope indicators and multipoint vertical extensometers. Contour maps of vertical and horizontal displacements can be derived and strains can be obtained from them. Using this strategy, Eisenstein, El-Nahhas and Thomson (1981) presented volumetric strains, shear strains and plastic zones (reproduced in Figure 53) around a deep tunnel driven with a closed face mechanical TBM, allowing considerable loss of ground and stress reduction, through a softened glacial till. The picture emerging is classical: pronounced volumetric expansions (from 2 to 3%) aside the tunnel, maximum shear strain (from 1 to 3%) zones resembling slip lines growing from tunnel crown and floor towards ground surface. The plastic zones suggest shear bands formation not properly found but likely present.

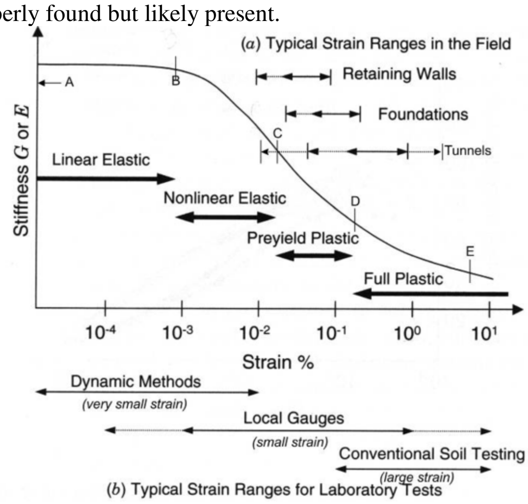


Figure 52. Stiffness degradation curve (modified from Mitchell and Soga, 2005): a- strain levels of typical geotechnical structures (modified from Mair, 1993); b- strain levels measured by laboratory tests (after Atkinson, 2000).

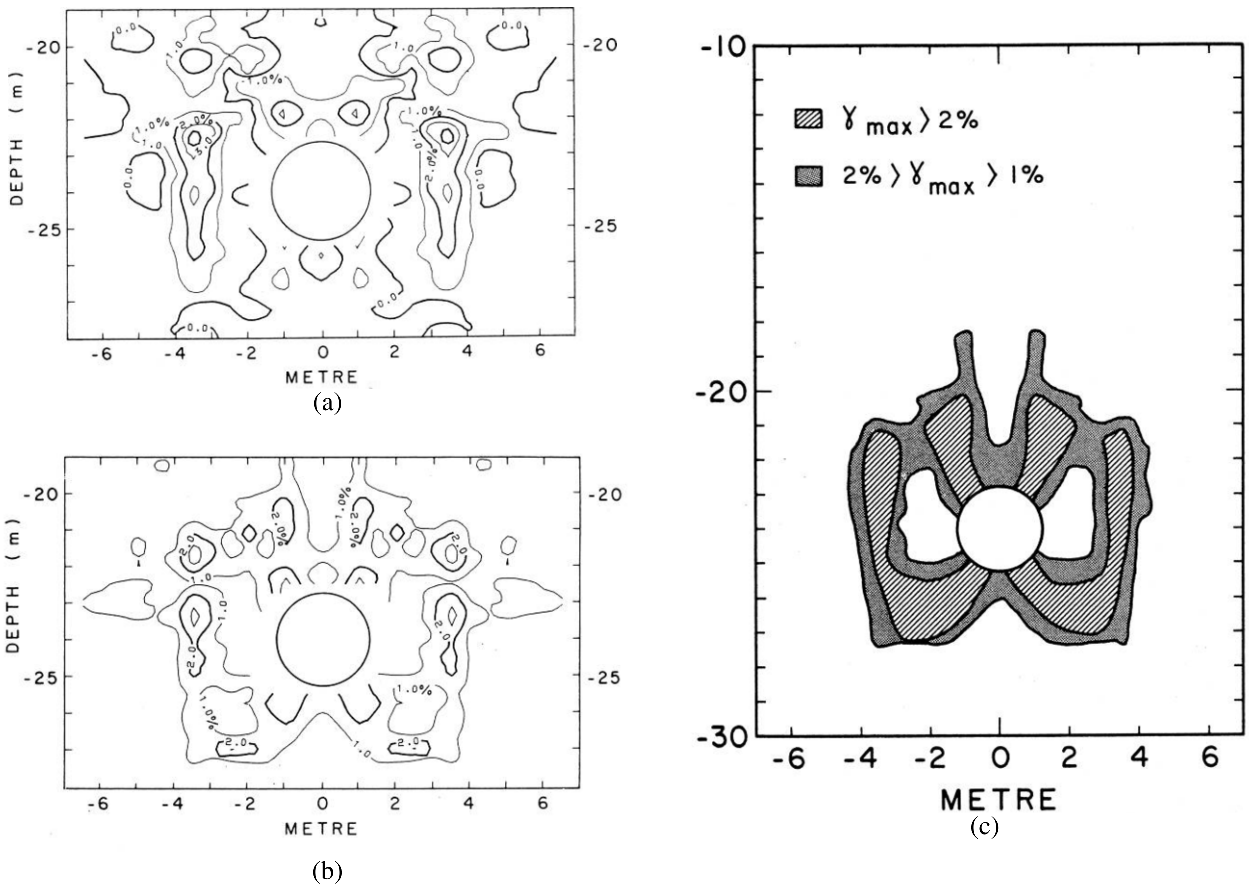


Figure 53. Contour maps of volumetric strains (a), maximum shear strains (b) and zones of mobilized shear strength (c) around a deep tunnel in till (modified from Eisenstein et al, 1981).

Ground stress changes by tunnelling induces volumetric and shear straining in the soil, that in prototypes can be inferred by conventional monitoring, including combined slope indicators and multipoint vertical extensometers. Contour maps of vertical and horizontal displacements can be derived and strains can be obtained from them. Using this strategy, Eisenstein, El-Nahhas and Thomson (1981) presented volumetric strains, shear strains and plastic zones (reproduced in Figure 53) around a deep tunnel driven with a closed face mechanical TBM, allowing considerable loss of ground and stress reduction, through a softened glacial till. The picture emerging is classical: pronounced volumetric expansions (from 2 to 3%) aside the tunnel, maximum shear strain (from 1 to 3%) zones resembling slip lines growing from tunnel crown and floor towards ground surface. The plastic zones suggest shear bands formation not properly found but likely present.

If the idealized stress and strain changes reviewed in Figure 51 now take place under undrained conditions, volume changes are inhibited and pore pressure changes occur. The short term pore pressure changes and the total stress distributions for those assumed condition, in a homogeneous, isotropic and non linear soil are illustrated in Figure 54, for a complete radial stress release at the opening. These plots can be obtained using Ladanyi (1966) numerical approach of non linear curve description of a collapsing cylindrical cavity.

Zones of positive and negative pore pressure changes develop as a function of the degree of over-consolidation of the soil as shown. If the in situ stress is only partially released at the opening, there will be a reduction in the magnitude of the pore pressure changes and zones of suction may disappear in both cases. Figure 55 presents the pore pressure changes of a soil element at the tunnel contour, as the internal tunnel pressure is decreased. Since the opening contour is assumed impervious, once the critical state is attained at undrained conditions, the deviator stress remains constant and the pore pressure changes thereafter are equal to the changes in the mean normal stress. Pore pressure changes are equal to the changes in the internal tunnel pressure, and the curves in Figure 55 become straight lines with negative unit gradients.

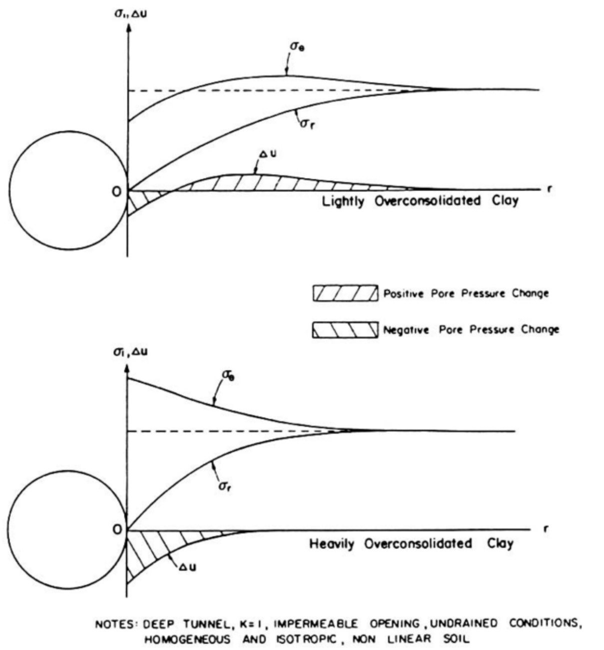


Figure 54. Short term pore pressure changes and total stress distributions around a deep tunnel.

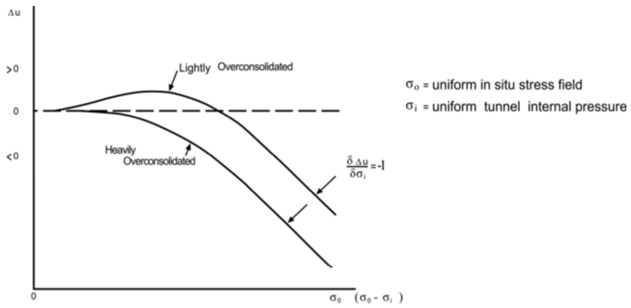


Figure 55. Pore water pressure changes in a soil element adjacent to the tunnel contour upon decrease of the internal tunnel pressure.

Now moving to differently idealized conditions, Figure 56 presents the schematic ground water flow net in the longitudinal plane containing the tunnel axis of a partially lined shallow tunnel, during construction through an incompressible and saturated granular soil. Tunnel heading is assumed to be stable under atmospheric air pressure; therefore the advancing excavation represents continuously changing hydraulic boundary conditions of the permeable ground domain. The transient hydraulic boundary conditions in an advancing heading may explain Laplacean pore water pressure changes solely controlled by the rate of tunnel advance, thus being time independent.

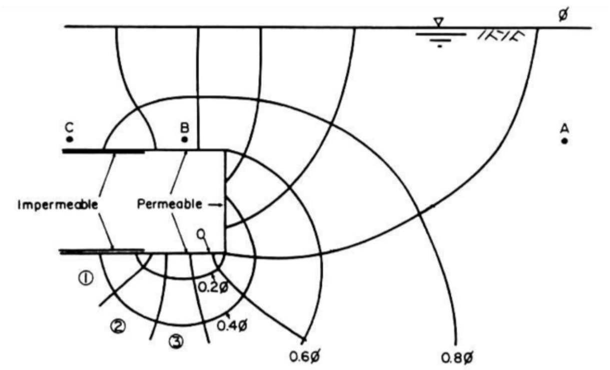


Figure 56. Schematic water flow around a partially lined shallow tunnel during heading advance.

After examining the pore water pressure changes in time dependent and in time independent conditions, it is apparent the need to identify which condition prevails in certain tunnel case where performance is being monitored. For this purpose, the criteria set by Negro and Eisenstein (1991) can be instrumental, though approximate. For a deep unlined tunnel with an impermeable contour, the linear elastic consolidation solution provided by Carter and Booker (1982) was used to obtain Figure 57, in which zones of negligible consolidation (average degree of consolidation $\bar{U}$ smaller than 10% or time factor $T = ct / ro^2$ smaller than 0.1) are mapped together with zones of appreciable consolidation ($\bar{U}$>90% or T>100) for given time intervals. On the other hand, Figure 58 shows similar solution for a shallow tunnel with a rigid permeable lining wished-in-place without changes in ground stresses, except the pore pressures that become zero at the tunnel contour. Along AB gravitational flow develops and, if one admits that the triangular excess of pore water pressure dissipation along AB is one dimensional, the consolidation of the soil elements in the tunnel cover along the symmetry line can be assessed by Terzaghi's theory. In this case, the time factor becomes $T = 4c/H^2$ due to double drainage. Once more, zones of negligible consolidation ($\bar{U}$ <10% and T<0.008) are mapped together with zones of appreciable consolidation ($\bar{U}$>90% and T>0.848) for given time intervals. Using these figures, one can estimate if drained or undrained conditions prevail in certain observed tunnel performance. Needless to say that considerable degree of judgement should be used with these or any other criteria for interpretation of performance of any real tunnel.

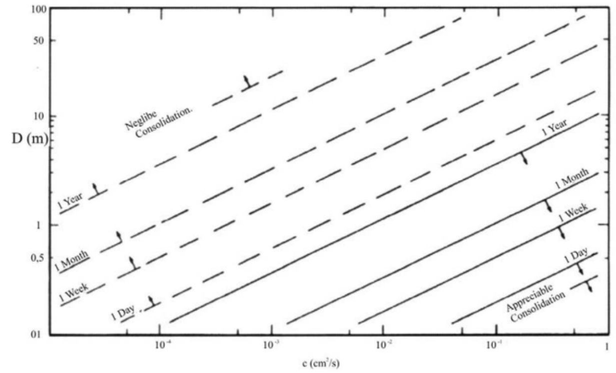


Figure 57. Drained and undrained responses around a deep and impervious tunnel (Negro and Eisenstein, 1991).

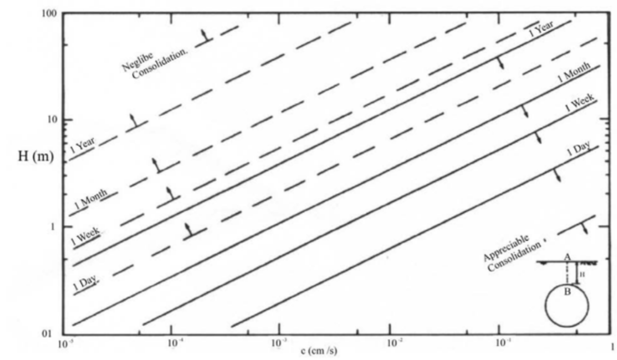


Figure 58. Drained and undrained responses in the cover of a shallow and pervious tunnel (Negro and Eisenstein, 1991).

When sizeable regions of the unsupported ground mobilize the soil strength and fail, mechanisms of collapse may form. It may be localized as roof collapse in dense sands or in stiff fissured clays or residual soils, without creation of mechanisms extending up to surface. These *local collapses* may or may not trigger a *global collapse*. In the latter, zones or bands of high shear concentration, such as those in Figure 53.c), may travel up to the surface and bounds a block of ground cover that slides into the unsupported heading, since in most instances the lining has sufficient capacity to inhibit collapse mechanisms in the supported sections. Except in cases of poor lining-ground contact, global instabilities are limited to the tunnel face or to the unsupported or partly supported heading, showing an entirely three-dimensional nature as in mechanisms A (face-roof instability, in short headings) and B (roof instability, in longer headings) shown in Figure 59.

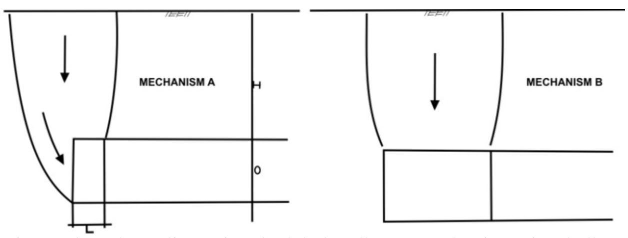


Figure 59. Three-dimensional global collapse mechanisms in shallow tunnels.

The role of the plane strain stability with time of a tunnel in which the lining action is represented by an internal tunnel pressure was discussed by Negro and Eisenstein (1991). Figure 60 depicts possible changes in average shear stress, pore pressure and factor of safety of a shallow tunnel in over-consolidated and in normally consolidated clay with time.

Tunnel construction is assumed to be undrained with pore pressure redistribution occurring thereafter, with shear stresses remaining constant. Depending on the stress release allowed, on the over-consolidation ratio and on the final equilibrium pore pressure (u_f), the pore pressure in over-consolidated clay may or may not reach a minimum at the end of construction (u_{oc}). Thus, long term stability may or may not be critical (Figure 60.a).

If tunnel construction in normally consolidated clay induces low shear stress mobilization (Figure 60.b), the pore pressure may increase by the end of undrained construction (u_{nc}) and then decrease with time (if u_f is smaller than u_{nc}). Stability will be critical in short term. For poorer ground control and higher shear mobilization, pore pressure change becomes negative and a critical long term stability condition may prevail. It is known from observations and theory that if ground control is good, the undrained changes in pore pressures around the tunnel are likely to be small, compared with other geotechnical structures. In contrast to an open cut excavation, the mean principal stress in the tunnel cover does not decrease as much. Pore pressures are mainly control by shear stress changes. If these are limited, as in good tunnelling practices, the changes in the factor of safety after undrained construction are small, provided tunnel contour is impermeable.

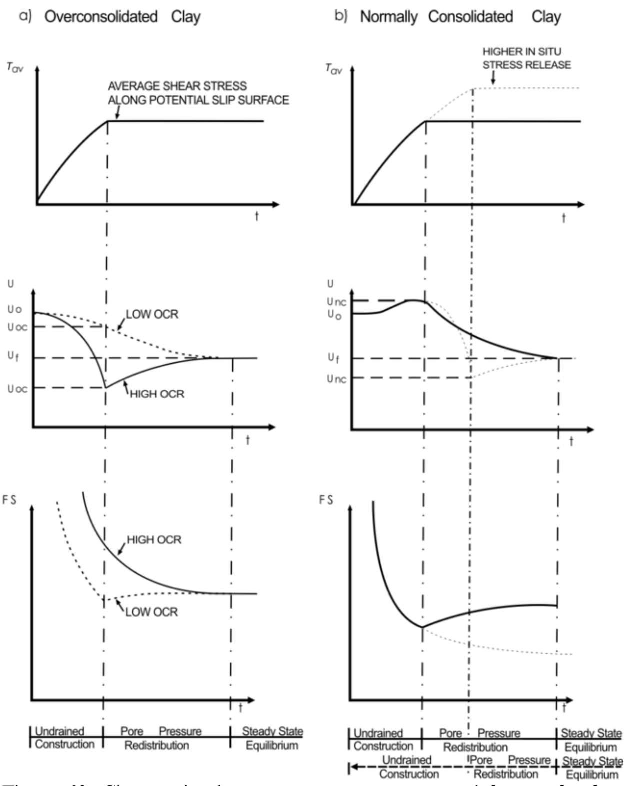


Figure 60. Changes in shear stress, pore pressure and factor of safety during and after construction of a tunnel: a) in over-consolidated clay, b) in normally consolidated clay (Negro and Eisenstein, 1991).

5.2 *Measuring the Performance of Tunnels in Soil.*

Reviews on monitoring the performance of tunnels were presented by Leca et al. (2000) and by Guilloux and Kastner (2001). The benefits and principles of geotechnical instrumentation are discussed in section 6 of this report. Broadly speaking, for any geotechnical structure, there are three essential questions to be answered when planning field monitoring systems. They are: ***why?***, ***what?*** and ***how?*** to monitor a geotechnical structure. There is however some specificity when answering these general questions for a soil tunnel project. This is quickly covered below.

There are two sets of answers for the standard question "***why measuring soil tunnel performance?"*** and they depend on what type of tunnel construction technology is involved. For a traditional mining construction (NATM[1], for instance), the main reasons to measure tunnel performance are:

a) the assessment of the critical stability condition of the ground mass (treated or untreated), prior to the lining activation, at the tunnel face or at the unsupported heading (usually, after proper lining installation, the stability condition of supported ground mass improves);
b) the assessment of the tunnel construction impact on the environment and of the potential damage on neighbouring structures or utilities;
c) the possibility of interactive design, by virtue of which the lining installation and/or the ground conditioning can be optimized, or the tunnel advance rate can be increased or else the possibility of inverse analysis, for parameters assessment and possible re-design of tunnel.

For a TBM driven tunnel, the common reasons for measuring tunnel performance are:

i) the assessment of the efficiency of tunnel face stability control;
ii) the assessment of the efficiency of grouting behind lining for loss of ground control;
iii) the assessment of the tunnel construction impact on the environment and of the potential damage on neighbouring structures or utilities;
iv) the possibility of interactive design, by virtue of which TBM operation and/or slurry or foam parameters are optimized.

An inverse analysis for ground parameters assessment from the performance of a tunnel built with modern TBM technology, with slurry or in an EPB construction mode, is almost an unlikely venture, due to the increased number of variable operational construction parameters that affect ground performance.

There are also two basic sets of answers for the second standard question ***"what is to be measured in soil tunnels?"*** regardless the construction technology involved:

1) displacements in the ground, as they can be related to the safety of the excavation fulfilling conditions for (a), (i) and (ii) above;
2) displacements, distortions and loads in the lining or in existing structures, fulfilling conditions for (b), (c), (iii) and (iv) above.

Regarding the answer to the third standard question ***"how to measure displacements and loads for tunnels in soil"***, briefly, there are five sets of standard mostly used instruments suitable for soil tunnel monitoring:

A) surface monuments and settlement points in structures or in the lining, traditionally measured with accurate optical levelling, for precise settlements measurement or, more recently, with less precise total stations, including robots, for three-orthogonal displacement components measurement;
B) subsurface settlement point, measured with accurate and precise optical levelling and magnetic anchor measured with magnetic sensors coupled with less precise survey tape or with a precise mechanical extensometer;
C) inclinometer grooved pipes, vertically or horizontally installed, traditionally measured with servo-accelerometers probes, furnishing displacement components transverse to the pipes along the grooves;

[1] Despite recognizing that the acronym NATM is truely inadequate to refer to sprayed concrete lining tunnelling method (Kovari, 2001), its use will be retained herein for the sake of brevity, with no offensive intention.

D) lining convergence measurements with invar tapes, cables or bars, furnishing precise relative lining movements;
E) load cells for pre-fabricated segmented tunnel linings, contact pressure cells for soil lining interfaces, strain meters for reinforcing steel bars, stress release techniques with precise and accurate measurement of induced lining strains, for back estimates of lining loads, piezometers for pore water pressure changes measurements.

In section 6 of this report fibre optic sensing applications are reviewed. These available technologies can yield novelties in soil tunnel monitoring, replacing any instrument of the five set standard group of devices listed above, with considerable advantages. Recent advances in fibre optic technology for geotechnical monitoring were reviewed by Mair (2008). Some new instruments are already available: SOFO strain sensors supplied by Smartec, EFPI pressure sensor by Roctest, FBG single-axis accelerometer by Fibre Sensing, SOFO inclinometer by Smartec, etc. Examples of application of these instruments in tunnels are given by Glisic and Inaudi (2007).

Beside these products, which are fully developed and commercially available, one could conceptually propose new devices. Though not granting its feasibility, one could conceive a new geotechnical inclinometer as it follows. It could be based on distributed Brillouin scattering sensors, possibly including a reference fibre. With them one could take advantage of existing pipeline monitoring know-how to develop a ***FO Inclinometer***.

A prototype of such inclinometer could use Smartape or Smartprofile strain sensors applied at 0, 90, 180 and 360 degrees over a 100 mm HDPE pipe, adequately protected, mimicking the grooves layout of an inclinometer casing. For a standard strain resolution of $20 \cdot 10^{-6}$ m/m and at a spatial resolution of say 1500mm, one could measure rotations larger than 1:1,666 in the A and B orthogonal directions, which is a resolution better than a standard slope indicator probe.

This prototype could be horizontally installed parallel to the axis of an urban shallow tunnel in soil, prior to its construction, using a robust and precise horizontal directional driving drilling system, at a certain depth below surface, in lengths of a city block (from 100 m to 200m).

Care should be taken to ensure that the A and B directions are vertical and horizontal. Else, supplementary monitoring of strains could be devised to assess possible pipe spiraling and associated torsion straining. After installed, the inclinometer should be grouted with a weak clay cement mixture, to ensure adequate fixture to the surrounding soil and assure conformance in terms of strains to be measured during the tunnel advance.

Differences in strains in the pipe walls measured in the vertical plane at a certain position can furnish the pipe vertical rotation at this position, during tunnel advance and this rotations can be integrated from the zero strain region, ahead the tunnel face, back to the tunnel or to the inclinometer pipe starting point and this would yield to the soil settlement profile along tunnel axis. As shown in section 5.4, the change in sign of the derivative function of this curve, which can be easily obtained, is an indicator of tunnel face instability. Similar approach applied to opposing sensors in the horizontal plane would furnish the ground lateral movement.

Alternatively the ***FO Inclinometer*** pipe could be installed vertically, aside the tunnel to be built, in order to measure ground movements laterally or longitudinally to the tunnel, during its advance.

The pipe in this lay-out should be installed down below the tunnel, to a depth where zero ground movements can be assumed, likewise a standard inclinometer casing.

Figure 61 illustrates the conceptual ***FO Inclinometer*** in the two lay-outs described and furnishes some details of the proposed design. Redundancy offered by the four distributed Brillouin sensor assures the knowledge of spatial bending of the tube through the average curvature. The average straining of the tube provides information on its longitudinal traction or compression, when installed parallel to the tunnel and as the tunnel heading advances. The unstrained reference fibre used allows absolute strain measurements. If temperature sensors are included (Brillouin scattering sensors can provide temperature measurement) as is the case of Smartprofile, the inclinometer horizontally installed can detected risk of blow-outs, if (warmer) air pressure is lost ahead the tunnel face, or if excessive (also warmer) slurry or water flow is established ahead the TBM face, possibly creating excessive ground heave.

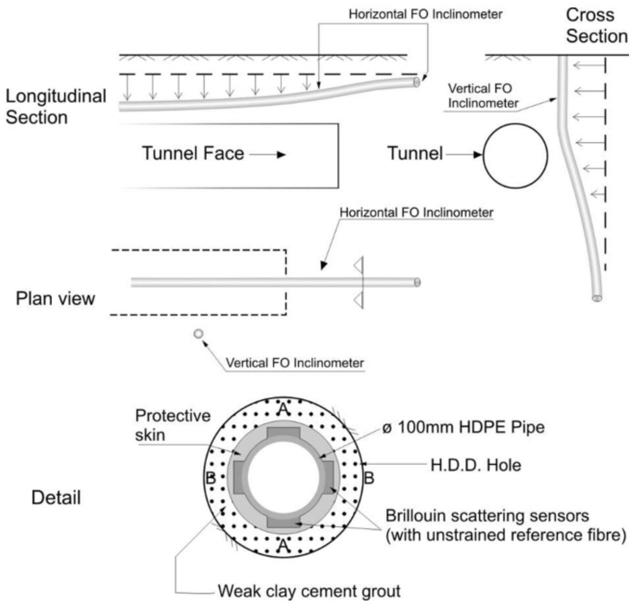


Figure 61. Conceptual FO Inclinometer installed around a tunnel in soil.

5.3 *Evaluation of Prediction*

A standard requirement for geotechnical structures observation is the need of a point of view. Without it you may not observe the performance adequately (in the right place, with the appropriate instrument, with correct accuracy). Therefore, for monitoring you need an estimate of the performance, a prediction of any kind, from semi-empirical to numerical. Negro (2009) reviewed the practice, in Brazil, for designing urban tunnels in soil. Figure 62 presents how practitioners estimate settlements routinely in that country and Figure 63 shows how they assess lining loads in plane static systems. It is believed that an international survey which is being currently prepared by TC28 on the same subject may reveal similar results: a large preference for numerical methods (finite elements or finite differences) for prediction of ground displacements and lining loads. It seems thus justified the need of numerical *modelling for monitoring* tunnel performance.

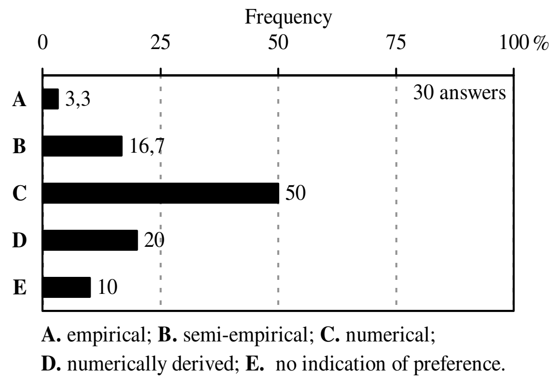


Figure 62. Preferred methods for settlements estimates in Brazil (Negro, 2009).

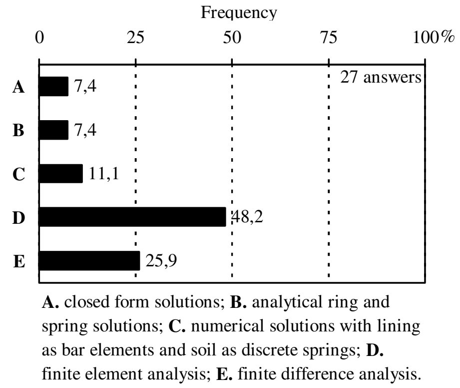


Figure 63. Preferred methods for lining loads assessments in Brazil, using 2D static systems (Negro, 2009).

Comparisons between prediction and performance is an accepted and standard option for detection of unconformities in the performance, which might be related to non conformities in the construction (with respect to design specifications) or to non conformities in the design (deficiencies in the tunnel modelling proper, in the ground behaviour representation or in the assessment of ground variability). Notwithstanding this, straight comparisons and analysis of deviations between expected and actual prototype behaviour are not always the only or the most appropriate way to detect non conformities and to anticipate required changes in the project to achieve its aim.

Difficulties with *modelling for monitoring* will be further covered in this report, but it seems worth referring to the work by Schweiger (1998), also presented in Carter et al. (2000), on bench marking numerical evaluation for tunnels in soil. Schweiger (op. cit.) submitted two modelling problems to numerical analysts of tunnelling, from academy and industry, both referring to a certain plane strain mined tunnel, which used sprayed concrete as lining.

In the first problem, excavation was performed in just one step, full-face mode, with a prescribed stress relaxation, followed by lining installation. In the second problem, a heading and bench excavation was represented for the same tunnel, with partial installation of the sprayed concrete lining, first in the heading and then in the bench and invert, using two values for the linear elastic stiffness of the concrete, on account of its hardening, and varying ground stress release. Linear elastic perfectly plastic model with associated Mohr-Coulomb failure criterion was prescribed with given parameters for soil. A temporary invert was not specified for the heading support and it appears that, as a consequence, a near collapse condition was met before complete tunnel support, which seemed to have numerically conditioned the results of some analysis. The ill-nature of the second problem prevented sensible results interpretation. Ten analysts provided results for the first problem, with three out of them using stiffness reduction rather than the required stress reduction for representation of the actual 3D stress transfer process, in the plane strain model and their results should have been discarded. The remaining seven results showed similar but not coinciding values in terms of surface settlements, of maximum normal forces and maximum bending moments, the latter exhibiting higher discrepancies. Not apparent from Schweiger (op. cit.) work is how tunnel profile geometry, with composed circular arches, was furnished to the analysts. Bending moments are highly dependent on the lining profile geometry and on how it is represented in a finite element or a finite difference analysis (beams versus multiple rows of continuum elements, number of rows of elements and number of elements along tunnel contour). The differences found (of as much as 50% in bending moments) could partly be attributed to these factors. Despite this, and agreeing with Poulos el. al. (2001), who also reviewed bench marking numerical evaluation, the exercise discussed indicates that numerical modelling of geotechnical structures such as tunnels, does require guiding and training in order to achieve reliable solutions, particularly when defining limiting values for field monitoring.

Two additional aspects may be added to the discussion on the representativeness of *modelling for monitoring* interpretation. Firstly, it seems convenient to stress the need to "*design the tunnel modelling*": this includes the mesh design and the constitutive representation of the ground as discussed by Potts et al. (2002) on geotechnical structures in general and by Pang et al. (2005) on tunnel modelling specifically. The time saving with modern and efficient pre and pos processing routines, which are available in most commercial computer programs for stress-strain analysis, allows more time to design the analysis, though unfortunately, not many users reckoned this or find it relevant.

The other aspect to be considered in the discussion is the *uncertainty* involved in the tunnelling performance and the proper way to handle it in the *modelling for monitoring*. Uncertainty is partly due to *variability* of the ground. The classic way of handling natural variability is through probabilistic approaches. Considerable advances have been reported from researches in this field. Peschl (2004) successfully treated finite element modelling of a tunnel with PLAXIS 2D, using *random sets* of ground parameters and defined serviceability and ultimate states in terms of ground surface displacements, through an optimized yet involving numerical strategy. Song et al. (2005) studied the effect of spatial distribution in geotechnical properties on results of tunnel modelling with FLAC 2D. Assuming a Gaussian distribution for geotechnical parameters, these authors have shown that a covariance of 40% on strength or on Young's modulus can increase tunnel crown settlement up to 10 or 16% respectively.

Despite the advances on probability approaches applied to tunnels in the academy, industry still favours deterministic analysis, using averaged soil properties and accounting for variations of parameters by using appropriate factors of safety. Reasons for this were addressed by Ralph Peck in 1995, quoted by Whitman (2000): "(…) *Practitioners* (dealing with traditional problems such as tunnels) *have not readily adopted reliability theory, largely because the traditional* (deterministic) *methods have been generally successful and engineers are comfortable with them. In contrast, practitioners in environmental geotechnics and to some extent in offshore engineering require newer, more stringent assessments of reliability that call for a different approach.* (…) *It is not surprising that those engineers working in environmental and offshore problems should be more receptive to new approaches* (such as reliability methods)*, and it should not be surprising that there may be spillback into the more traditional areas."* Part of the "comfort" referred to by Peck is related to the fact that traditional methods avoid the higher costs involved in geotechnical investigations required to cover the stochastic description of the ground and to cover geological uncertainties. Also, they avoid the higher costs associated to the still time consuming probabilistic numerical modelling. Alternative, simpler and less expensive approaches for routine tunnelling problem analysis may follow the approach presented by Duncan (2000) and may also come with new and more efficient modelling tools that we shall see in the near future. But whatever development we may have in the future tunnelling practice, it will always be an approximation, ever requiring sound engineering judgement for its use.

Having these aspects in mind, it seems convenient to review published comparisons between numerical predictions and performance of tunnels in soft ground. This material has an inherent bias as it tends to show the best results of comparisons made, since authors usually avoid making public poor results. Negro and Queiroz (2000) reviewed results of 65 published comparisons made from 1977 to 1998. This review is extended herein, by adding comparisons made after that period, totalling more than one hundred cases reviewed. Table 10 presents a complementary and comprehensive list of comparisons made in the last decade, clearly not attempting to be exhaustive and retaining the same structure and criteria used before.

Table 10. Review of some numerical predictions of shallow tunnels using numerical modelling.

	Author/Year	Origin	Tunnel	Ground	Construction Method	Pred. Type		Numerical Simulation	Anal.	C.M.S.	Surface Settlement			Subsurf. Settl.		Horiz. Displ.		Lining Loads		R. L.
66[1]	Addenbrooke et al. 1997	UK	Jubilee Line Extension	Stiff Clay	Shield	C1	y	Imp. Conv.	2D-FE	NLEPy[4]	<	≡	G	-	-	-	-	-	-	1
67	Benmebarek et al. 1998	France	Lyon Metro Line D	Silty Soils	Slurry Shield	C1	y	Imp. Conv.	2D-FD	EPf	<	-	R	≡	G	>	G	-	-	2
68	Almeida e Sousa 1998	Portugal	Sao Paulo Metro	Tropical Lateritic Porous Clay	NATM	C1	x	3-D	3D-FE	NLEPy[5]	≡	≡	G	≡	G	<	R	-	-	2
69	Almeida e Sousa 1998	Portugal	Sao Paulo Metro	Tropical Lateritic Porous Clay	NATM	C1	z	Stress Red.	2D-FE	NLEPy[5]	≡	≡	G	>	R	≡	R	-	-	2
70	Conceicao et al. 1998	Portugal	Mato Forte	Marlly Limestone	Mined	B1	y	Stress Red.	2D-FE	EPf	≡	<	G	-	-	<	G	-	-	2
71	Martins et al. 1998	Portugal	Porto - Tunnel 4	Granitic Residual Soil	NATM	C1 ?	x	3-D	3D-FE	EPf	≡	≡	G	-	-	-	-	-	-	1
72	Bakker et al. 1999	Netherlands	2nd Heinenoord Tunnel	Holocene's Sands and Clays	Slurry Shield	C1	y	Imp. Conv.	2D-FE	EPf	-	-	-	-	-	-	-	>	R	1
73	Bakker et al. 1999	Netherlands	2nd Heinenoord Tunnel	Holocene's Sands and Clays	Slurry Shield	C1	y	3-D	3D-FE	EPf	-	-	-	-	-	-	-	>	R	1
74	Benmebarek et al. 1999	France	Lyon Metro Line D	Alluvial Clays and Sands	Slurry Shield	C1	y	Imp. Conv.	2D-FD	EPf	≡	<	R	-	-	≡	G	-	-	2
75	Lee et al. 1999	U.K.	DLR Lewisham Extension	Woolwich & Reading Beds	Slurry Shield	C1	y	Stress Red.	2D-FE	EPy	≡	≡	G	-	-	-	-	-	-	1
76	Dias et al. 1999	France	Line 2 Cairo Metro	Alluvial Sand	Slurry Shield	C1	y	3-D	3D-FD	EPf	>	-	-	-	-	-	-	-	-	1
77	Dias et al. 1999	France	Line 2 Cairo Metro	Alluvial Sand	Slurry Shield	C1	y	Imp. Conv.	2D-FD	EPf	<	-	-	≡	G	-	-	-	-	1
78	Tang et al. 1999	U.K.	Heathrow Express Trial	London Clay	NATM	C1	x	3-D	3D-FE	EPf	<	<	G	>	G	-	-	-	-	1
79	Gioda & Locatelli 1999	Italy	"Monteolimpio 2" Italy-Switzerland	Alluvial Sand Deposit	NATM	C1	y	Stress Red.	2D-FE	E	<	≡	R	-	-	>	R	-	-	2
80	Dias et al. 2001	France	Lyon Metro Line D	Silty Soils	Slurry Shield	C1	x	3-D	3D-FD?	EPf	≡	≡	R	-	-	-	-	-	-	1
81	Faria et al. 2001	Brazil	Brasilia Metro	Porous Clay	NATM	C1	z	3-D	3D-FE	EPy	≡	≡	R	≡	R	-	-	-	-	1
82	Wu et al. 2001	Germany	High-Speed Line Cologne-Frankfurt	Weathered Sedimentary Rock	NATM	B1	x	Stress Red.	2D-FE	EPf	≡	≡	G	-	-	-	-	-	-	1
83	Bakker et al. 2009	Netherlands	2nd Heinenoord Tunnel	Holocene's Sands and Clays	Slurry Shield	B1	z	3-D	3D-FE	EPf	≡	≡	G	-	-	-	-	-	-	1
84	Koelewijn et al. 2009	Netherlands	2nd Heinenoord Tunnel	Holocene's Sands and Clays	Slurry Shield	C1	x	3-D	3D-FE	EPf	>	>	R	-	-	-	-	>	R	3
85	Hededal et al. 2001	Denmark	Copenhagen Metro	Limestones	NATM	B1	y	Stress Red.	2D-FD	EPf	>	>	R	-	-	-	-	-	-	1
86	Sato et al. 2001	Japan	Diversion Channel	Dense Sands with Fines	Slurry Shield	A	x	Stress Red.	2D-FE	E	>	≡	R	>	R	≡	R	-	-	2
87	Jordan et al. 2002	Spain	Marin Pontevedra Rail	Weathered Granite and Gneiss	Mined	B1 ?	x	Stress Red.	2D-FD	EPf	>	≡	P	-	-	-	-	-	-	1
88	Melo & Pereira 2002	Portugal	Line 2 Shanghai Metro	Alluvial Clays	EPB Shield	C1	z	3-D	3D-FE	EPf	≡	≡	G	≡	G	≡	G	>	R	4
89	Hu et al. 2002	Germany	Nuremberg Metro Line U3	Keuper Sandstone (crumbly rock)	NATM	B1	x	Stress Red.	2D-FE	EPf	≡	-	-	≡	-	-	-	-	-	1
90	Hu et al. 2002	Germany	Nuremberg Metro Line U3	Keuper Sandstone (crumbly rock)	NATM	B1	x	Stress Red.	2D-FD	EPf	≡	-	-	≡	-	-	-	-	-	1
91	Oota et al. 2005	Japan	Osaka Metro - Site A	Soft Alluvial Clay	Slurry Shield	C1	z	Imp. Stress	2D-FE	EPf?	<	<	R	-	-	≡	G	-	-	2
92	Akutagawa et al. 2005	Japan	Railway Rokunohe	Cohesive Sandy Soils	NATM	C1	x	Stress Red.	2D-FE	E	<	<	R	-	-	-	-	-	-	1
93	Akutagawa et al. 2005	Japan	Railway Rokunohe	Cohesive Sandy Soils	NATM	C1	x	Stress Red.	2D-FE	EPf	<	<	R	-	-	-	-	-	-	1
94	Akutagawa et al. 2005	Japan	Railway Rokunohe	Cohesive Sandy Soils	NATM	C1	x	Stress Red.	2D-FE	EPf6	≡	≡	G	-	-	-	-	-	-	1
95	Hoefsloot & Verweij 2005	Netherlands	Sophia Railway	Pleistocene Sand	Slurry Shield	C1	y	3-D	3D-FE	NLEPy7	>	<	P	-	-	<	R	-	-	2
96	Barla et al. 2005	Italy	Torino Metro Line 1	Sands and Gravels	EPB Shield	C1	y	Stress Red.	2D-FD	EPf	≡	≡	R	-	-	-	-	>	-	3
97	Moller & Vermeer 2005	Germany	Steinhaldenfeld	OC Marl	NATM	C1	y	Stress Red.	2D-FE	EPf	≡	<	R	-	-	-	-	-	-	1
98	Moller & Vermeer 2005	Germany	Steinhaldenfeld	OC Marl	NATM	C1	x	3-D	3D-FE	EPf	≡	<	R	-	-	-	-	-	-	1
99	Grasso et al. 2005	Greece	Twin Driskos	Siltstone-Sandstone	NATM	B1	y	Stress Red.	2D-FE	EPf6	-	-	-	>	-	>	-	-	-	2
100	Teparaksa 2005	Thailand	Premprachakorn	Stiff Bangkok Clay	EPB Shield	C1	w	Stiff. Red.?	2D-FE	EPf	≡	<	R	≡	G	-	-	-	-	1
101	Teparaksa 2005	Thailand	Bangkok Subway	Soft and Stiff Clay	EPB Shield	C?	w	Stiff. Red.	2D-FE	EPf	>	>	R	-	-	-	-	-	-	1
102	Pang et al. 2005	Singapore	MRT North-East Line	Granitic Residual Soil	EPB Shield	C1	x	3-D	3D-FE	EPy	≡	<	G	-	-	-	-	>[8]	G	3
103	Foa et al. 2005	Brazil	Salvador Metro	Gneissic Residual Soil	NATM	C1	x	3-D	3D-FE	EPy	>	-	P	-	-	-	-	-	-	1
104	Marques et al. 2006	Brazil	Brasilia Metro	Porous Clay	NATM	C1	x	3-D	3D-FE	EPy	≡	≡	G	≡	G	≡	G	>	R	4
105	Eclaircy-Caudron et al. 2007	France	Bois de Peu	Marls and Limestones	Mined	B1	y	Stress Red.	2D-FE	EPf	-	-	-	<	-	<	-	-	-	2
106	Abu-Krisha 2007	Egypt	El-Azhar Road	Slightly Silty Sand	Slurry Shield	C1	x	3-D	3D-FE	EPf	>	-	P	-	-	>	R	-	-	2
107	Tong et al. 2007	China	West Mao Mountain	Sandy Clayey Gravel	NATM	B1	y	Stress Red.	2D-FE	EPf	-	-	-	≡	-	-	-	-	-	1
108	Yoo et al. 2007	Korea	Multiple Seoul Metro	Granitic Residual Soil	NATM	C1	x	3-D	3D-FE	EPf	-	-	-	>	R	-	-	-	-	1
109	Liang et al. 2008	China	Thunder Bay	Silty and Sandy Soils	TBM	C1	x	3-D	3D-FD	EPf	≡	≡	G	≡	G	>	G	-	-	2
110	Shahin et al. 2008	Japan	2D Model	Aluminium Rods	Self Weight	B1?	x	Imp. Conv.	2D-FE	EPy	≡	≡	G	-	-	-	-	≡[9]	G	3

1. For cases 1 to 65, see Negro and Queiroz (2000).
2. Type of prediction according to Lambe (1973) classification. The question mark indicates a certain degree of uncertainty; x: actual prediction; y: back analyses; z: prediction with previously calibrated model; w: any of the former x, y or z.
3. Abbreviations: Pred. : prediction; Anal. : Type of Analysis; C.M.S. : constitutive model for soil; Subsurf. Settl. : subsurface settlement; Horiz. Displ. : horizontal displacements; Mg. : maximum magnitude; Dist. : maximum distortion; Dtr. : overall distribution; R.L. : Rank Level for the comparisons; E: Linear Elastic model; NLE Nonlinear Elastic model; NLEPy Nonlinear Elastic Plastic model with distinct yield and failure surfaces; EPf: Elastic Plastic model with yield and failure surfaces coinciding; EPy: Elastic Plastic model with distinct yield and failure surfaces; Core Removal: Progressive Core Removal; Stress Red. : Ground Stress Reduction; Imposed Conv. : Imposed Tunnel Convergence; Stiff. Red. : Ground Stiffness Reduction; Imp. Stress : Imposed Stress; ≡: calculated value approximately equal to measured value; >: calculated value greater than measured value; <: calculated value smaller than measured value; G: good; R: regular; P: poor; FE : finite element; FD : finite difference; 2 and 3D : two and three dimensional.
4. Anisotropic ICFEP. 5. Lades's model. 6. With strain softenning. 7. Hardening model. 8. Linning loads in adjacent pile. 9. Earth presure.

Data from the previous review is not included in Table 10, though they are taken into consideration in the statistics that follows. Readers are referred to Negro and Queiroz (op. cit.) for earlier data and references. One notes that 3D analyses are becoming popular, yet mainly in the academy, as this type of modelling is still "engineering time consuming", as coined by Möller and Vemeer (2005), thus less prevalent in industry. Case numbering follows the sequence of the earlier review. The table includes reference and year, country where comparison was made, tunnel project name, prevalent ground type, tunnel construction method, type of prediction according to Lambe (1973) classification (see Table 11). Following Negro and Queiroz (op. cit.), a further qualification was appended to Lambe's classification (see also Table 11) by, whenever possible, identifying if the case was an actual prediction (x), a back-analysis (y), a prediction done using previously calibrated model (z) or a not clearly identified class (w). Furthermore Table 10 provides indication on how the numerical simulation was done, particularly how 3D effects were accounted in the 2D analyses. The type of stress-strain relation used for the ground is also given. Lining representation was not always provided, but in general, lining material was assumed to be linear elastic and for the interface lining-ground, a no-slip condition was assumed.

Table 11. Classification of prediction, according to Lambe (1973): appendage by Negro and Queiroz (2000).

Prediction	When it Made	Results
A	Before event	-
B	During event	not known
B1	During event	known
C	After event	not known
C1	After event	known

Negro and Queiroz (2000) appendage:
(x): actual prediction
(y): back-analysis
(z): prediction with previously calibrated model
(w): not clearly identified case

Finally, Table 10 provides a qualitative comparison between calculated and measured performances, including: a) surface settlements: magnitude of the maximum settlement and of the maximum distortion transverse to the tunnel plus a subjective evaluation of the transverse settlement trough; b) subsurface settlements: magnitude of the maximum subsurface settlement and an overall assessment of the distribution of settlements with depth; c)horizontal displacements: magnitude of the maximum transverse ground displacement and an overall appraisal of the distribution of transverse displacements with depth; d) lining loads: magnitude of the maximum acting loads in terms of radial stresses onto the lining, or thrusts or, in few cases, bending moments. Negro and Queiroz (op. cit.) contended that a proper comparison between prediction and performance should be thorough, in the sense it ideally has to include comparisons of the complete ground displacement field and lining loads. In fact, it is much simpler to match the settlement trough than the entire ground displacement field and lining loads. Accordingly, the four levels comparison rank provided by Negro (1998) and reproduced in Table 12 was adopted as a further qualification of comparison made: the lowest Level 1 applies to comparisons involving just one performance aspect and the highest Level 4 applies to those involving all performance aspects considered. Appropriately, Table 10 includes the rank level of each case study reviewed. The calculated values is said to be equal to the measured value whenever the latter is not more than 10% greater and not less than 10% smaller than the former. Otherwise the calculated value is said to be greater or smaller than the measured value. The spatial distributions of displacements or loads were arbitrarily defined as good, regular or poor, after a liberal comparison between prediction and measurement was made.

Table 12. Rank Levels for comparisons between prediction and performance of tunnels in soil (Negro, 1998).

Comparison Level	Vertical Displacements		Horizontal Displacements		Lining Loads
1	X	or	X	or	X
2	X	and	X		
3	X	or	X	and	X
4	X	and	X	and	X

Likewise the earlier review, most of the cases analysed do not provide sufficient information regarding the case history, including construction and monitoring details, and in-depth details of the numerical simulation. Regarding the first difficulty, that results from space limitations imposed on most publications, tunnel experts are invited to use a novel publishing web space recently made available by Technical Committee 28, under the auspices of the University of Science and Technology INSA Lyon, France (http://tc28.insa-lyon.fr/) in which they can input their complete case history field data with no relevant space restrictions. The lack of details in the case history or in the analysis can alone hinder any careful assessment of the prediction. Notwithstanding this, once again likewise the earlier review, an attempt was made to identify the efficiency of the modelling tools used, related to soil type, construction method, type of prediction, of numerical simulation, of constitutive model used. Once more no clear correlation was found between those factors and the certainty of the prediction. This difficulty, as earlier, allowed only a broad appraisal to be attempted in what follows.

Figure 64 shows the accumulated frequency of published comparisons over the last 30 plus years being noticeable an increase in the yearly publication rate. This may reflect an increasing interest with the subject and the increasing availability of accessible modelling tools.

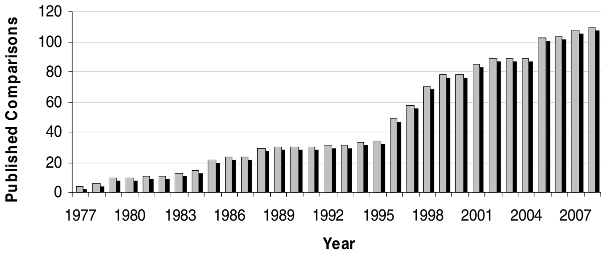


Figure 64. Accumulated frequency of published performance comparisons with time.

Figure 65 shows the distribution of origin of these studies dominated by Europe and Asia, as before, being apparent a decline of contributions from North and South America over the last decade.

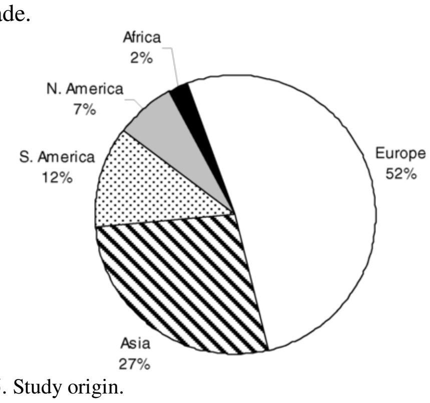


Figure 65. Study origin.

Figure 66 reveals that the great majority of the cases involve soils with cohesive strength component, condition possibly favouring a tunnel option and possibly rendering also a more treatable modelling condition.

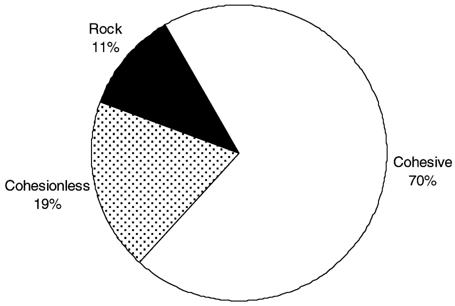


Figure 66. Types of ground involved.

Traditional mining methods tend to dominate the cases studied (see Figure 67). These tunnel construction methods tend to be more instrumented in the field, offering chance to more frequent comparisons to be made but TBM excavated tunnels became more popular in the last decade. Some classes of the latter, present features of construction such as pressure at the face and around the shield body, with EPB or slurries, plus grouting pressures behind the lining, which tend to be more complex to be properly handled and represented in a numerical simulation, especially in a 2D model.

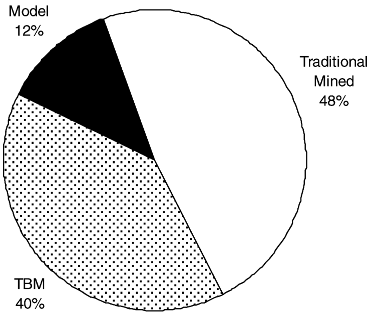


Figure 67. Construction methods involved.

As shown in Figure 68, the majority of cases (76%) refer to predictions type C1, made after the event, with results from field instrumentation already known. Note also that most of C1 type cases refer to back-analyses (y, with 27% of cases).

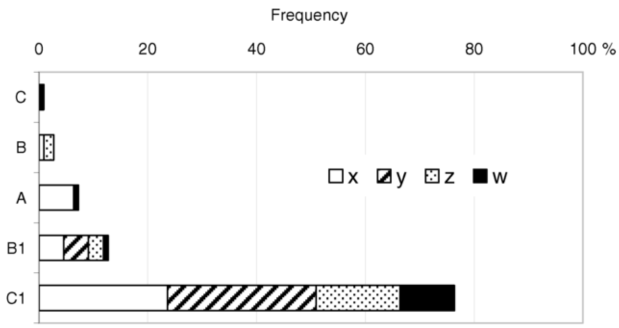


Figure 68. Types of prediction according to Lambe (1973), with appendage by Negro and Queiroz (2000).

The most frequently modelling tool used is still the two dimensional finite element analyses (Figure 69). However, over the last decade the use of 2D FE declined, this followed by an increase in 2D finite difference analysis and by a quite substantial increase in the use of 3D Finite element analysis, mainly in the academy, but also in industry, thanks to affordable and improved hardware and software.

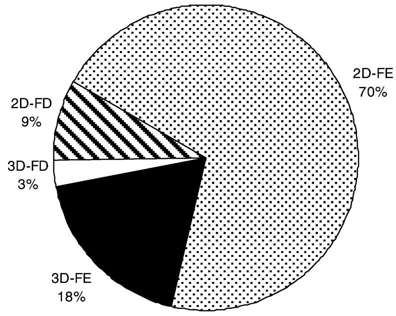


Figure 69. Types of numerical analysis performed.

With respect to the account of the 3D effects of the tunnel advance in a 2D numerical representation (see Figure 70), the most used procedure is the ground stress reduction, accounting for 70% of the cases of 2D analysis reviewed.

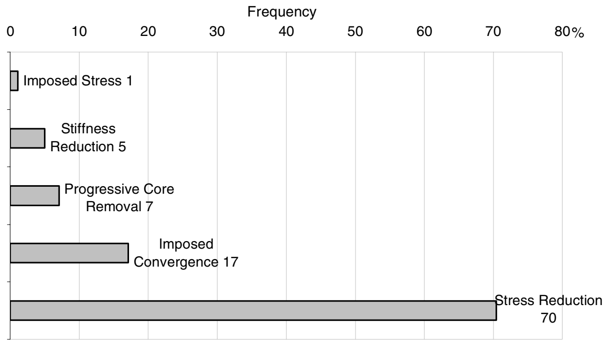


Figure 70. Account of 3D effects on 2D analysis.

Regarding the stress-strain model adopted for ground representation (see Figure 71), preference is given to linear elastic-plastic models, in which the yield and failure surfaces coincide (EPf). These are seconded by elastic-plastic models with non coinciding yield and failure surfaces (EPy), models developed mainly by geotechnical engineers. In fact, most of the commercial numerical codes available have in built constitutive models of the former type.

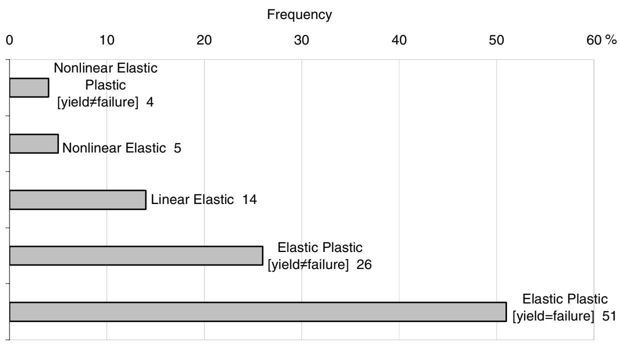


Figure 71. Types of stress-strain models used for ground.

More than half of the comparisons refers to Level comparisons (Figure 72), in which only one tunneling performance aspect is investigated, most frequently surface settlements. Level 4 comparisons, where the entire tunneling performance is compared, were performed in 12% of the cases, and a considerable and unjustified decline of Level 4 comparison was noted in the last decade. Clearly, whenever just one aspect of the performance is addressed in the comparison, the prediction, or even the back-analysis, becomes more flexible and more easily adjustable to the observations, particularly those of types B1 and C1.

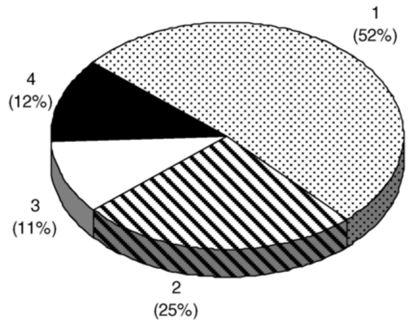


Figure 72. Rank level of comparisons.

The results of comparisons between calculated and observed surface settlements are shown in Figure 73. The magnitude of the maximum observed surface settlement is closely matched by numerical prediction in just more than 60% of cases, a surprisingly poor result, having in mind the possible bias of the data covered, as explained earlier in this section and considering that in the majority of the cases the maximum settlement was known (B1 and C1 cases mainly). Over-prediction and under-prediction are observed in similar proportion. The overall distribution of surface settlements, however, is generally good or regular and seldom poor, as expected.

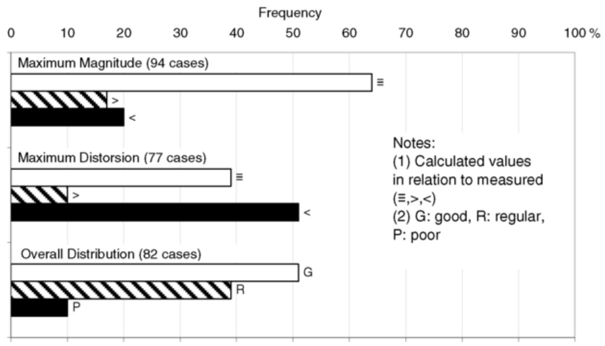


Figure 73. Calculated and measured surface settlements.

Regardless the type of analyses or the type of stress-strain model used, in more than half of cases the numerical simulations furnished distortions smaller than those observed. Some authors (Mair, 1979, Eisenstein, 1982, for instance) suggested that this is related to the concentration of shear strains into relatively narrow zones. It appears that the noted deficiency is related to the inability of most numerical methods to portray properly shear band formation. The trend to underestimate surface distortion may be compensated in practice by the usual conservative assumption of flexible building.

Figure 74 summarizes comparisons between calculated and observed subsurface settlements with depth. As shown, the maximum magnitude of deep settlement tends to be matched by the numerical analysis. Having in mind the concentrated shear straining that occurs around shallow tunnels in soil, one reckons that linear elasticity will hardly provide a good match of overall displacement distribution with depth. Note also that the highly frequent linear elastic-plastic analysis with coinciding yield and failure are reduced to simple elastic numerical modelling, if the ground control conditions met in the case histories are good, as they should in any urban tunnel projects, and the ground stress release is minimized. The usually good overall distribution of subsurface settlements shown in Figure 74 is likely due to the fact that only in very few cases field measurements were taken at points close to the tunnel, where displacement gradients are higher. The noted agreement is, broadly speaking, good at points at distances further than 20% of tunnel diameter. If measurements were taken at closer distances a not so good agreement would have been seen.

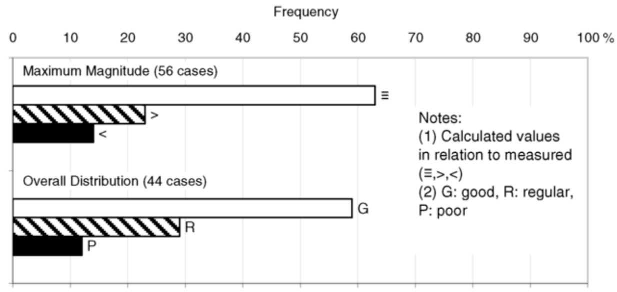


Figure 74. Calculated and observed subsurface settlements.

Regarding the maximum magnitude of the measured horizontal ground displacements transverse to the tunnel, these are matched or over-predicted by the numerical models (see Figure 75). As noted by Negro and Queiroz (2000), this feature could possibly be attributed to the procedure used to represent the 3D effects in the 2D analysis. In 2 dimensional ground stresses reduction simulation, which is the most frequent technique used to mimic the actual 3D stress transfer, a constant amount of stress release, defined as a fixed fraction of the in situ stresses, is applied to all points of the tunnel contour. Since this is an approximation, one could contend that different amounts of stress reduction along the tunnel perimeter could possibly improve the prediction of the maximum magnitude of lateral displacements and also improve the estimated distributions of lateral movements, which were found to be merely regular (see Figure 75). Though still limited in number, more recent comparisons with 3D numerical simulation results seem to support this view (see Melo and Pereira, 2002 and Marques et al., 2006): it appears that results of 3D analysis tend to show better agreement with measured lateral ground movements. Comparisons between results of 3D and 2D analyses could be used to define a new stress release criterion for 2D analysis in practice.

Different amounts of stress release around the tunnel contour may also have a positive effect on the overall distribution of lining loads, otherwise assessed simply as regular, as shown in Figure 76. The comparison of calculated and observed maximum magnitude of lining loads reveals similar trends to those noted for lateral ground movements: lining loads tend to be over predicted or matched by the numerical models and only rarely underestimated likewise lateral ground movements. No simple reason is found to explain these findings. Difficulties in measuring lining stresses or contact pressures are known, particularly with concrete stress cells (Dunnicliff, 1988) though considerable improvement has been achieved with pressure cells in sprayed concrete linings (Clayton et al. 2002). None of them, however, justify an alleged systematic under measurement that would explain the trend above. On the modelling side, most cases reviewed took no special account regarding the representation of the lining-ground interface. It is known that a lower strength interface (full or partial slip) reduces the maximum magnitude of lining loads estimated in prefabricated lining. For sprayed concrete linings most analysis used a simplified representation of concrete hardening and no account was taken regarding creep of the early age sprayed concrete. The normally adopted design simplification (the use of a reduced equivalent Young's modulus for sprayed concrete) is duly conservative thus leading to higher lining loads. The creep of concrete when loaded at early age leads to lining and ground relaxation, which in turn leads to lower lining loads eventually measured.

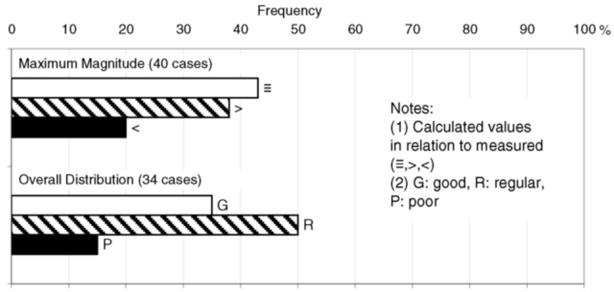


Figure 75. Calculated and observed lateral ground movements.

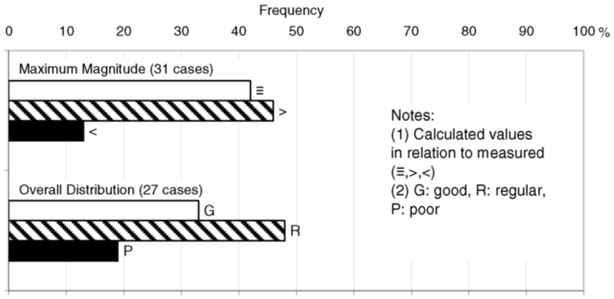


Figure 76. Calculated and measured lining loads.

Similarly to the previous review (Negro and Queiroz, 2000), no clear relation was found in the current one, relating quality of prediction and features of the numerical modelling. However, over the last decade, an increased use of 3D FE analysis was noted and some yielded the best ranked results reviewed (see Table 10 case 88, by Melo and Pereira 2002, on a slurry shield and case 104 by Marques et al. 2006, on NATM). There is a clear need of less biased evaluations of type A prediction, preferably of rank level 4. Comparisons of this group are still rare and should be preferred and encouraged, to compensate the dismaying believe of some, particularly regarding pressurized tunnelling, that prediction of performance of EPB and slurry shields are too much dependable on machine operation to render the tunnelling performance predictable (Shirlaw, 2000). Though not sharing such a radical view, otherwise not confirmed by results of continued research on the basic ground response to pressurized tunnelling (see for instance Bezuijen and van Lottum, 2006), the authors understand that there are considerable limitations in using comparisons between numerical predictions and measured performances to anticipate deviations of behaviour or non conformities in tunnelling.

Finally, one important aspect of tunnel performance, even more rarely investigated, is pore pressure generation and dissipation. Prediction of pore pressure and water flow was identified recently (Negro, 2009) as the least satisfactory area of tunnelling practice in Brazil. It is therefore auspicious the work carried out in Cambridge on modelling long-term response to tunnelling in clay, comparing calculated and measured pore pressures and ground vertical movements (Wongsaroj et al. 2007), though remain to be seen the combined results of horizontal ground movements and of lining loads with time, for a thorough evaluation.

5.4 *Evaluation of performance.*

Evaluation of performance is usually done by straight comparison of field measurements with predicted quantities. Among the latter, displacements are normally favoured for being simpler to measure. The design can define both limiting displacements for serviceability and ultimate state. The use of limiting displacements for assessment of performance of shallow tunnels has, however, some shortcomings. This is particularly true for stiff to hard ground masses, in which a near ultimate state condition may be reached with ground displacements of few centimeters, frequently raising undue skepticism on the validity of a given limiting displacement. This was the case of some documented tunnel failures in which reduced magnitude limiting displacements were exceeded just prior to collapse. Table 13 illustrates some of these cases, all of them built by the so – called NATM.

Table 13. Maximum surface and crown settlements measured prior to the collapse of some NATM tunnels.

Project	Itaquera (Brazil)	S. Amaro (Brazil)	Heathrow (U.K.)	Pinheiros (Brazil)
Year	1989	1993	1994	2007
Location	Sao Paulo, Brazil	Sao Paulo, Brazil	Heathrow, U.K.	Sao Paulo, Brazil
Equivalent Diameter (m)	8.5 (heading crown)	7.8 (heading crown)	7.50	14.50
Cover (m)	23.00	9.00	20.0	20.00
Excavated Ground Type	Hard clay	Stiff to hard silty clay and dense sand	Hard grey London clay	Foliated gneissic rock
Surface Settlement before collapse (cm)	0.8	3.4$^{(1)}$	5.5	6.3$^{(1)}$
Crown settlement before collapse (cm)	2.7	3.5	6.0	3.4
Reference	Sozio et. al., 1998	This report	HSE, 2000	This report

Note: $^{(1)}$ Includes settlements due to drainage.

Cording et al (1971) proposed for performance evaluation of underground rock caverns the ratio *measured to calculated elastic displacement at opening contour.* Whenever this ratio is smaller than 2 the opening is assumed to be stable and when it is greater than 5 to 10, modifications on the support and on the excavation methods was required to avert major failures. Kuwajima and Rocha (2005) presented a type A prediction (according to Lambe, 1973) for displacements around the Pinheiros Metro Station, Sao Paulo, Brazil, a shallow rock cavern, with a thin rock cover, below residual and sedimentary soils, that eventually collapsed during its construction (see Assis et al, 2008 and Barton, 2008). Kuwajima and Rocha (op. cit.) performed a 3D sequential finite element analysis, assuming for the rock a linear elastic plastic behaviour with a non associated plastic flow to the Mohr-Coulomb failure criterion. It appears that the underground station lay-out and construction sequence considered in the analysis was somewhat different from that finally built, but the main components were quite similar. The authors noted in the results of the analysis that no plastic zones were formed in the rock mass, which behaved essentially as a linear elastic ground. The authors have found maximum settlements of the rock cover of the order of 9 mm. If one follows Cording et al (op.cit) suggestions, for measured settlements smaller than 18 mm, the cavity would have been essentially stable. Instabilities were to be expected for deep settlements greater than 45 to 90mm. The station collapsed on the 12 of January 2007 just after the field instrumentation measured an accumulated settlement of 34 mm in the rock cover (see Table 13 above), a 'gray zone' value, larger than the lower limit offered but smaller than the upper limit proposed by Cording et al (1971) criterion. Putting aside the representativeness of the ground parameters used in the analysis, the reasons to question the applicability of this criterion for performance evaluation of this particular case are few, including the very low cover of rock above the opening and the structurally controlled non isotropic rock mass scenario.

This case illustrates some of the difficulties associated to performance evaluation through comparisons between measured and limiting calculated displacements. Other difficulties refer to the fact that in some instances the calculated displacements may not be fully reliable or may not be readily available. Displacement velocity is sometime used, in some geotechnical structures, to assess whether a stable or an unstable condition will be attained in time (see Sullivan, 1993 on pit slopes, for instance). Tunnels are structures with rapidly changing boundary conditions in space and in time, conditions not rendering adequate application of such criterion.

Dimensionless displacement such as deformation should be favoured for performance evaluation. The relation between factor of safety and deformation in geotechnical structures is well known. In embankment dams it is customary to refer to a limiting rate of horizontal movements per metre rise of fill, as a controlling parameter (Penman, 1986). It appears that *dimensionless quantities* derived from displacements as well as from other variables, can operate better for generalizations and calibration with the practice. Moreover, *redundancy of evaluation* is required due to the very nature of the assessment, involving complex ground conditions as well as complex boundary conditions. Accordingly, a review is presented herein of ***performance indicators*** for tunnels in soil, some of them related to serviceability, some to ultimate state, some already in use and some other not quite S_o. Their applicability and limitations are discussed.

a) Limiting crown settlement to tunnel diameter ratio (S_c/D).

Reviewing results of static and centrifuge tunnel model tests from Cambridge University in reconstituted soils (normally consolidated and over consolidated kaolin, dense and loose sand) and examining data from real tunnel collapses in soil (some provided in Table 13) one may conclude that near failure condition is attained for total crown settlements of 3 to 15% of the tunnel diameter. In other words, ultimate state is attained in soil tunnelling for ***S_c/D>0.03 to 0.15.*** This is an inconveniently wide range of values that may only be of some assistance together with other performance indicators. For most tunnels serviceability is jeohardized for S_c/D exceeding 3 to 4%.

b) Limiting surface to crown settlement ratio at tunnel axis (S_s/S_c).

At failure, with full development of vertical shear surfaces running from tunnel heading to the surface, the indicator ***S_s / S_c*** tends towards ***1.0,*** as the ground cover prism slides along vertical shear bands in an ultimate state condition. Perhaps more appropriately, the ratio of settlements increments (ΔS_s / ΔS_c) would tend to unity in the vicinity of failure. Far from failure, this ratio varies widely depending on the relative depth of the tunnel (cover H/D), on the in situ stress ratio (K_o), on the soil strength (c_u, Phi), and on the amount of ground stress release prior to the lining installation.

Figure 77 presents how the indicator surface to crown settlement ratio, at tunnel axis, varies in a plane strain condition, modeled by a *frictionless constitutive soil model*, with homothetic stress strainhyperbolic relationships (setting Janbu, 1963's exponent equal to zero), using finite element analysis, for variable tunnel cover to diameter ratios, variable undrained cohesive strength and variable amount of ground stress release, for K_o equal to one. Fairly complex relations of this ratio with the other variables are noted. Figure 78 presents similar results for a *cohesionless constitutive soil model,* for variable K_o and a fixed tunnel cover to diameter ratio. A similar degree of complexity of the function is observed once more.

It appears that this complex behaviour explains the scatter of data noted in Figure 79, reproduced from Ward and Pender (1981), in which are plotted two extreme curves obtained with the cohesionless soil model, for a friction angle of 20º and for 50% of in situ stress release, for the two indicated values of coefficient of earth-pressure at rest.

Moreover, caution should be taken when applying this indicator to consolidating (soft clays) or to contracting soils (loose sands, loess and porous clays) upon tunnelling: in these cases surface settlement may get closer or exceed the tunnel crown settlement, leading to an indicator ratio closer (or greater) than unity, without involving the collapse of the tunnel heading.

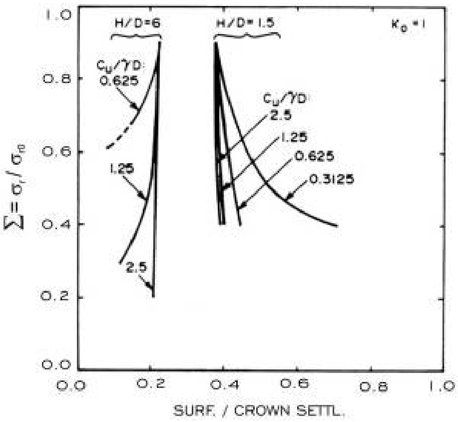


Figure 77. Changes in the indicator maximum normalized surface settlement with the amount of stress release and other variables, from FE calculations with the hyperbolic frictionless soil model.

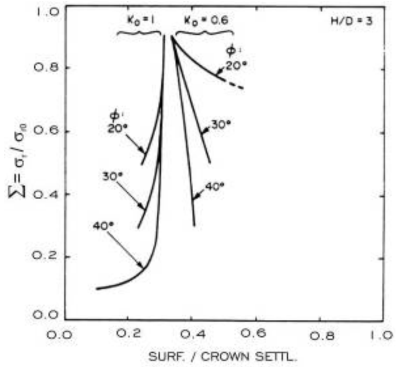


Figure 78. Changes in the indicator maximum normalized surface settlement with the amount of stress release and other variables, from FE calculations with the hyperbolic cohesionless soil model.

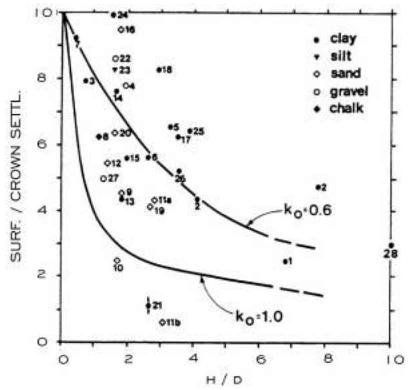


Figure 79. Normalized settlement ratios observed in some tunnels case histories (modified from Ward and Pender, 1981).

c) Limiting surface and subsurface distortions (γ).

Distortions in the ground induced by tunnel excavation may produce damage on existing structures at the surface or in the subsurface, in a similar way to that discussed in Section 5 on supported excavation. Distinctions between damages induced by a supported open cut and by a tunnel excavation refer to the magnitude of the horizontal strains involved in each case, which combined with the angular distortion affects the degree of damage imposed on a structure (Branco et al, 1990 and Namba et al, 1999). These indicators may refer to serviceability as well as to an ultimate state not of the tunnel proper but to the nearby structure being investigated. Ultimate ground distortions, however, may correlate with maximum shear strain in the ground associated to a near *tunnel* failure condition. Results from static and centrifuge tunnel model tested to failure may serve this purpose. Inspecting results of tests conducted in Cambridge, one may note that ***subsurface distortions*** at points close to the tunnel crown, in excess of ***1/10*** may indicate near tunnel collapse conditions in softer or looser soils, whereas values in excess of ***1/30*** may already render ultimate conditions in stiffer or denser soils. It should be noted, however, that these limits derived from tests in reconstituted and homogeneous soils and that non homogeneity of actual soils may radically change them. Recall also that the limiting distortions given derived from displacements measured at model markers spaced 50 to 10 mm apart. Actual shear straining is concentrated in shear bands of much smaller thickness, that may correspond to higher distortions than provided. Note also that the distortion indicator can be calculated from plots of ground settlements with distance to the advancing tunnel face. In so doing, one is calculating a virtual distortion rather than a real one, as it is derived from the difference of settlements at a given point, at distinct occasions, thus at different distances of the fixed given point to the changing position of the tunnel face with time. In this case, the distortion so defined is that for a 'uniformized' ground condition corresponding to that found at the given point where the settlement point was installed.

d) Longitudinal distortion index (LDI).

Negro and Kochen (1985) and Horiuchi et. al (1986) independently developed a criterion based on the longitudinal distortion distribution, which is the derivative of the settlement distribution ***u (x)*** along the tunnel axis either at surface or at any elevation in the tunnel cover. The index is given by:

$$LDI(x) = \partial u(x)/ \partial x \tag{6}$$

This is a real ground distortion interpreted as a shear strain, which relates to the slope of the longitudinal settlement profile, defined by a series of settlements points at surface or at subsurface, or by horizontal in-place inclinometers, or by settlement profilers. The index can be interpreted as a measure of the shear strength mobilization of the soil. Therefore, it may be related to the tunnel stability condition, allowing one to identify incipient collapse mechanisms. Figure 80(a) depicts the settlement and distortion distributions for a stable ground condition. The distortion distribution resembles a Gaussian normal probability curve. If some instability process is triggered around tunnel heading and face, a marked change in the shape of this curve is noted, even for a minor change in the shape of the settlement curve (Figure 80(b)). The *magnitude* of the maximum *LDI* could be taken as an index of ground control during heading advance by referencing it to some critical shear strain. For saturated clay, this strain would de $2c_u/E_u$ in which E_u is the undrained initial tangent deformation modulus. This criterion is similar to that proposed by Sakurai (1981), however it says little about overall stability of the tunnel, as it gives indication only of some localized ground failure that can prevail in non ultimate state condition.

The *distribution* rather than the magnitude of *LDI* furnishes a clearer indication of minor instability trends: the *LDI* decreases in some regions, perhaps becoming negative at some points and increasing somewhere else. Such changes in the distortion distribution pattern can be taken as a sign that a collapse mechanism is being formed. Kochen et al (1987) have shown that by using this indicator, it would have been possible to anticipate two days in advance the collapse of a subway tunnel in soft clay in Sao Paulo. Negro and Eisenstein (1991) have shown that this indicator agrees with results of 3D tunnel heading model tests in over-consolidated kaolin tested to failure in Cambridge. Just as a reference, Horiuchi et al (op.cit.) presented the case of a tunnel in medium uniform alluvial sand in Tokyo that collapsed with a maximum *LDI* at surface of 7% (***1/140***) whereas Kochen et al (op.cit.) found a maximum of 2.5% (***1/40***) for the collapsed tunnel in soft organic clay in Sao Paulo. Values for stiffer grounds are not available but are expected to be lower than the figures above.

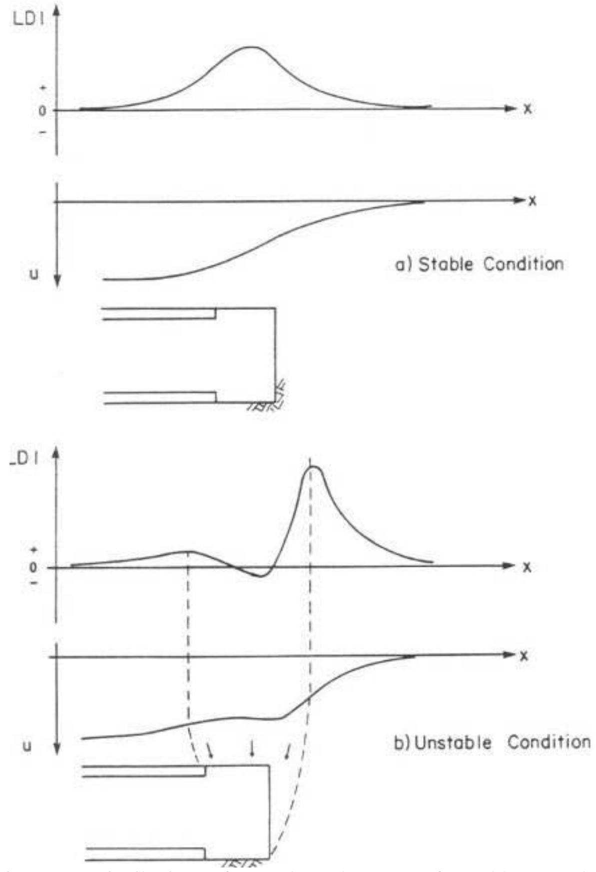
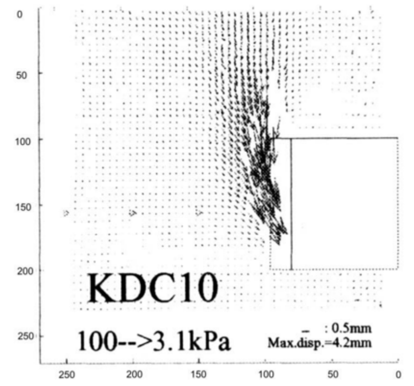


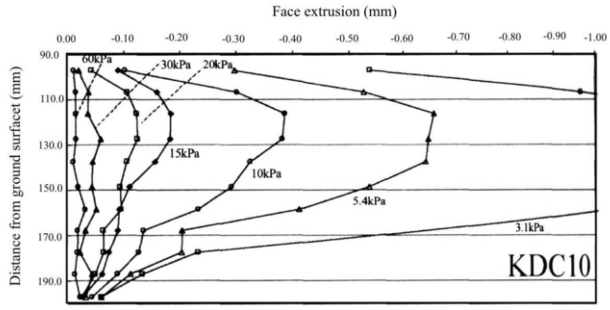
Figure 80. Distributions of LDI along the cover of a stable (a) and an unstable (b) tunnel.

Subjacent to this criterion is the fact that if a mechanism of collapse is being formed in the homogeneous ground mass just ahead the tunnel face, ground is being 'lost' into the tunnel (see item *f* ahead) and the volume of ground excavated per metre of tunnel is larger than the nominal cross sectional area of the tunnel. The increased flow of ground into the opening implies in changes on the surface distortion measured by *LDI* and, as corollary, it implies in changes in the ground displacement vector magnitude and orientation close and around the tunnel heading face. This is illustrated in Figure 81 taken from Date, Mair and Soga (2008). These are results of centrifuge tunnel model tests on dense sand with decreasing internal tunnel pressure. A considerable increase in the horizontal component of ground displacement vectors close by tunnel face is noted when approaching failure.

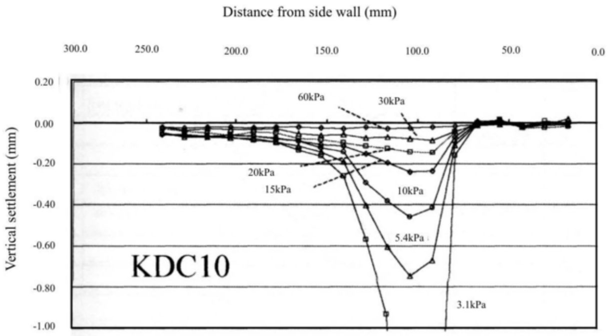
For the condition above, in a traditional mined tunnel, with sprayed concrete lining, one would note a change in the orientation of the lining displacement vector measured by total station at points of the tunnel contour close to the face. As collapse is approached, the ***displacement vectors orientation change*** at these points showing *increasing angle to the vertical* against the direction of excavation. For homogeneous ground and for relatively deep tunnels, this is a sign of impending collapse as much as a change on the *LDI* distribution indicates instability. Note, however, that in non homogeneous ground, the changing orientation of displacement vector is related to the tunnel approaching zones of contrasting stiffness as major faults or dikes crossing a rock tunnel. This effect was noted and explored by Schubert and Budil (1995) as an element of performance anticipation for deep tunnels. Figure 82 taken from Grossauer, Schubert and Sellner (2005) illustrates this effect.



(a)



(b)



(c)

Figure 81. Displacement vector changes with the decrease in internal tunnel pressure (a), face extrusion (b) and subsurface settlements (c) in centrifuge tunnel model test (modified from Date et al, 2009).

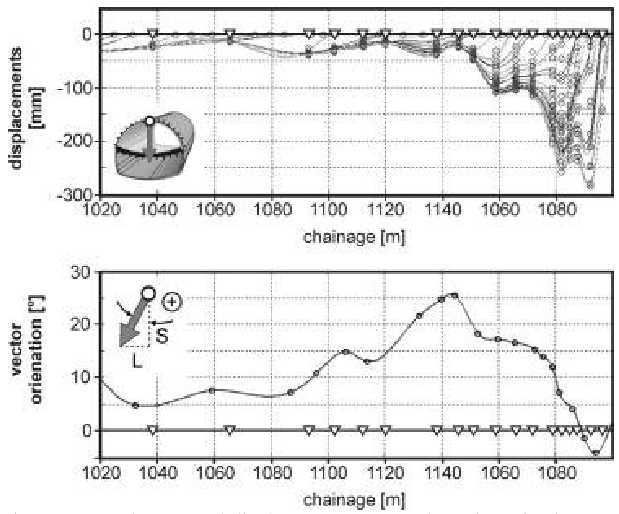


Figure 82. Settlement and displacement vector orientation of points at a deep tunnel contour as a fault is approached (modified from Grossauer et al, 2005).

e) Volume of surface settlement (%V_s).

The volume of the transverse settlement trough per metre length of tunnel as a percentage of the tunnel excavation area can be taken as an indicator of tunnel performance. It is usually assessed by fitting a normal probability curve through surface settlement measurements at sections normal to the tunnel. This volume is some time referred to as *volume loss* (see Mair and Taylor, 1997 and Standing and Burland, 2005), though the latter is better described as the amount of ground lost into the tunnel. Due to volume changes in the ground cover, they may not be the same. Herein the first meaning will be retained. Negro (1979) extended suggestions made by Peck, Hendron and Mohraz (1972), recognizing that both loss of ground and volume changes are directly related to the quality of construction and the amount of ground stress release which is allowed by the tunneling operation. Four levels of construction quality and volume of surface settlements were defined and are reproduced in Table 14.

Table 14. Construction quality and volume of surface settlements.

Construction Quality	Range of %V_S
High	< or = 0.5%
Normal	0.5 and 1%
Poor	1 and 3%
Pre-failure condition	3 and 40%

Though entirely empirical, this criterion was able to explain improved ground responses in tunnels built in Sao Paulo and Frankfurt (Heinz, 1984). A limitation of this indicator, however, is apparent from Figure 83 taken from Negro, Sozio and Ferreira (1996) for tunnels built in Sao Paulo. In it, a correlation between **%V_s** and the transverse distortion γ at surface is attempted, in order to relate the former with the degree of damage on surface structures, on the grounds that a good quality tunnel construction would likely represent smaller risks of damage. This is not clearly the case. Take a 'normal' construction quality corresponding to %V_s of o.5 to 1%. This range of settlement volume corresponded in Sao Paulo to surface transverse distortions ranging from 1:2000, which hardly would result in damaging structures, to 1:200, which may result in severe damages in buildings on surface. This shortcoming was pointed out by Branco el at (1990).

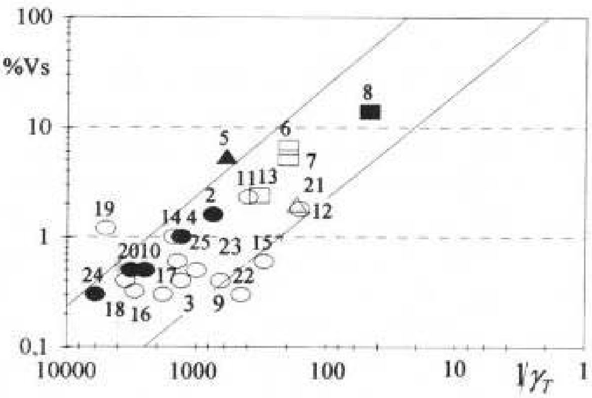


Figure 83. Volume of surface settlements and surface transverse distortions (from Negro, Sozio and Ferreira, 1996) (see original reference for identification of tunnel cases numbered).

f) Volume of soil lost (loss of ground, %V_l).

The volume of soil that displaces across the tunnel perimeter per metre length of tunnel expressed as a percentage of the tunnel excavation area is the volume of soil lost or ***loss of ground (%V_l)***. Cording and Hansmire (1975) suggested that this performance indicator could be expressed as a function of crown settlement S_c measured at a close distance y above the tunnel:

$$\%V_l = 100 \,.\, S_c \,.\, 2(R + y) / 2 \pi R^2 \qquad (7)$$

where $\boldsymbol{R}$ is the equivalent radius of tunnel excavation. Cording (1991) reviewed the sources of losses of ground over a shielded tunnel (face loss, shield loss, tail-lining gap loss and lining deflection loss). Mair (1996) and Mair and Taylor (1997) provided typical ranges of losses of ground for some construction technologies for certain ground scenarios, gathered from tunnel projects of late last century. These are reproduced in Table 15 and provide reference for serviceability.

This indicator operates reasonably well for assessing the ground control condition near the excavation, but says too little about its impact on the surface, which is dependent on the ground volume changes taking place in the tunnel cover. Volumetric expansion in the ground that takes place in dense sand reduces $\%\boldsymbol{V_s}$ but contraction observed in porous soils or in consolidating soft and compressible clays enhances the surface settlements.

Table 15. Typical losses of ground (modified from Mair and Taylor, 1997).

Technology	Ground type	Range of loss of ground (% V_l)
Open face tunnelling	Stiff clays	1 to 2
NATM	Stiff clays	0.5 to 1.5
EPB and slurry shields	Sands	>0.5
EPB and slurry shields	Soft clays	1 to 2
EPB and slurry shields	Mixed face	2 to 4

For most tunnels, serviceability is endangered for loss of ground in excess of 4 to 6% and a near ultimate state condition is achieved for losses greater than 8 to 10%.

g) Lining distortions (Δ D/D%).

Lining distortions, defined as relative changes in the tunnel diameter, are routinely measured by tape, cable or bar extensometers. Measuring points are also subjected to precise levelling. More recently total stations were introduced to measure absolute lining displacements with reduced precision but increased efficiency and smaller impact on construction operations. Schmidt (1984) put forward ultimate lining distortions ranges for different soil types which are reproduced in Table 16.

Table 16. Limiting lining distortions ratios recommended (modified from Schmidt, 1984)

Soil Type	Limiting Distortion Range Δ R/R (%)
Stiff to hard clay (OF < 2.5 – 3)	0.15 – 0.40
Soft clays or silts (OF > 2.5 – 3)	0.25 – 0.75
Dense or cohesive sands, most residual soils	0.05 – 0.25
Loose sands	0.10 – 0.35

Notes:
(1) Add 0.1 – 0.3% for tunnels in compressed air, depending on air pressure.
(2) Add appropriate distortion for external effects such as passing neighbouring tunnels
(3) Values assume reasonable care in construction, and standard excavation and lining methods.

These values were derived from field observations and are to be used for design verification of lining upon bending, following American design practice. However, they can be used for lining performance evaluation, noting that higher distortion values have already been reported under extreme circumstances. This was the case of precast segmented lining used in a Mexico City tunnel through soft soils, in which diameter changes as much as 6% were measured without support collapse (Schmitter and Moreno, 1983). Distortions larger than 1% were measured after lining invert closure, in lightly reinforced sprayed concrete primary support of Brasilia double track Metro tunnel, in soft porous clay, without signs of distress (Negro, 1998). It appears that the recommendations above derived from tunnel cases in which good ground control conditions prevailed. One important component of these conditions is a good lining and ground contact in which concentrated ground loading on lining is avoided and in which no voids exist between support and ground mass or were fully grouted. If deviations from these conditions exist, the recommended limiting distortions may not be valid.

h) Reference lining load (%Overburden).

Stress release techniques seem to be one of the most robust approaches for measurements of normal stresses in tunnel linings. Among these, the mini-flat-jack test is of particular interest for lightly reinforced sprayed concrete linings (see Kuwajima et al, 1991 and Negro, 1994). From the lining normal stresses one can derive the corresponding ground stresses acting onto the lining. Despite recent improvements (Clayton et al 2002), contact pressure cells still present difficulties to furnish reliable measurements of radial stresses on concrete linings.

Negro and Eisenstein (1997) discussed relations between delayed lining activation, measured in terms of distance from the tunnel face to the section where complete lining ring comes into full contact with the soil mass (usually within 1 to 2 tunnel diameters), and ground stress relaxation in shallow tunnels, defined as a percentage of the mean in situ principal stresses, in a plane strain condition transverse to tunnel, at the depth of the tunnel axis. The first author investigated these relations in a very large number of projects, finding out that, provided good ground control conditions were present, reflecting either good construction quality or simply good ground quality, ground stress release at lining activation varied broadly from 20 to 70% (Negro and Eisenstein, op. cit.) and final average ground stresses onto the lining corresponded to ***25*** to ***75%*** of the ground in situ stresses at tunnel axis. More frequently than not they represent ***50%*** of in situ stresses. This range of values can be taken as a limiting serviceability range of lining loads and whenever measurements lie outside this range, a non conforming condition may be present and further analysis and investigations should be undertaken, to identify and explain the noted performance.

i) Maximum lining load

Maximum lining loads are normally taken from the lining structural design which can define ultimate lining loads. As mentioned above, loads in the lining can be preferably assessed by stress release techniques applied to the lining installed. Stress-metres and strain-gauges installed in the lining or on its surface can also be used but present known difficulties and limitations (see Dunnicliff, 1988 and Kuwajima, et al op. cit).

For thin linings in weak soils under high ground stresses, buckling might be an issue. If the tunnel cross section is circular, the maximum uniform pressure causing collapse by buckling is given by Morgan's (1961) expression:

$$P = 3EI / R^3 + E_s / (1+\nu) \qquad (8)$$

in which E and I are the lining Young's modulus and moment of inertia respectively and E_s and ν are the elastic constants for the ground.

It should be noted that both the ultimate lining loads given by the structural design and the buckling pressure above can provide unsafe estimates of ultimate lining loads if the ground-lining contact is poor. If there are local concentrations of ground loads, ungrouted spaces or stable voids behind the lining, an ultimate state can be reached at lower ground stresses.

j) Dimensionless crown displacement (U_c).

Results of drained and undrained tunnel model tests in clays and sands carried out in Cambridge, in which relations between factors of safety and vertical crown displacements were assessed and measured, allowed Negro and Eisenstein (1991) to propose a limiting dimensionless crown displacement beyond which near collapse conditions are likely to prevail. The dimensionless displacement is defined as:

$$U_c = S_c \,.\, E_{co} / D\sigma_{cro} \qquad (9)$$

where: S_c is the radial displacement at the tunnel crown, E_{co} is the initial (in situ) tangent modulus of elasticity (for small strains) of the tunnel cover (at a point D/2 above crown), D is the tunnel diameter and $\boldsymbol{\sigma_{cro}}$ is the initial (in situ) radial stress at the crown. In drained conditions E_{co} is taken as a drained modulus and $\boldsymbol{\sigma_{cro}}$ is an effective stress. In undrained conditions E_{co} would be an undrained modulus and $\boldsymbol{\sigma_{cro}}$ a total stress. The introduction of a soil stiffness parameter reduces the test result scatter slightly, as it should: for a factor of safety of 1.5, the corresponding range of U_c was found to be 0.5 to 1.5, whereas the crown settlement to diameter ratio was 0.2 to 3.8%.

Inspection of the test results revealed that $\boldsymbol{U_c}$ values in excess of ***1.8*** will generally imply in near collapse condition, with full development of high shear strain concentrations. This would be a ***near ultimate*** limiting crown displacement. For so called good ground control conditions, where shear band formation is not noted, where tunnel serviceability is not jeopardized by excessive deformations (S_c/D smaller than 3 to 4%) and losses of ground are acceptable (up to 3%), the limiting ***serviceability*** crown displacement $\boldsymbol{U_c}$ will be typically equal to ***1.0*** or less (FS of about 1.5) Consequently under equivalent conditions and for the same factor of safety, a softer soil may experience larger crown displacements than a stiffer soil. Indeed, model tests results and actual tunnel cases show that final collapse is attained sooner, in terms of displacement magnitude for stiffer soils. The above criteria, both for serviceability and for ultimate conditions estimates, were tested in few dozens of well documented case histories of actual tunnel constructions and was proved satisfactory.

Two main limitations of the above criteria should be recalled. The first is the requirement to know the profile of the initial tangent modulus of elasticity, at the section where the crown measurements are being taken. The second refers to the stress path dependency of this modulus, complicating further its assessment. It is understood that the deformation modulus at play is mainly related to the ground cover of the tunnel. It is noted that for Sao Paulo sedimentary and residual soils, K_o ranges from 0.7 to 1.1 typically. The dominating stress path at tunnel cover is that of almost pure shear, with some rotation of principal stresses. It has been noted in a number of cases of tunnels built in that city, that an operational in situ tangent elastic modulus can be taken as $\boldsymbol{E_{co}}$***=5 to 6*** $\boldsymbol{N_{spt}}$ in MPa (Negro, Sozio and Ferreira, 1992).

On the account that the tunnel factor of safety is related to the transverse surface ground distortion (γ_T), Figure 84 reproduces a correlation between the latter and the dimensionless crown displacement, U_c, (see Negro, Sozio and Ferreira, 1996 for the numbered case histories references) from tunnels built in Sao Paulo: one can note that the ultimate crown displacement $\boldsymbol{U_c}$ of ***1.8*** corresponds to transverse surface distortions of ***1/250*** to ***1/1000*** and that for a serviceability crown displacement $\boldsymbol{U_c}$ of ***1.0*** corresponds to transverse distortions of ***1/500*** to ***1/2000***.

Table 17 reproduces dimensionless crown displacements at the four tunnel cases given in Table 13 prior to their collapses. The results seem to be in agreement with the correlation given above for ultimate state.

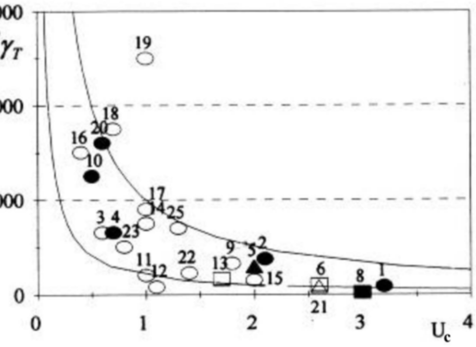


Figure 84. Maximum transverse surface distortions and dimensionless crown displacements in tunnels built in Sao Paulo (from Negro, Sozio and Ferreira, 1996).

Table 17. Dimensionless crown displacements at some tunnels prior to collapse.

Project	Itaquera (Brazil)	S. Amaro (Brazil)	Heathrow (U.K.)	Pinheiros (Brazil)
Ground cover	Hard sandy clay	Stiff silty clay	Hard grey London clay	Residual soil, gneiss saprolite
Estimated E_{co} (MPa)	300	60	200	1000
Crown settlement before collapse (mm)	27	35	60	34
U_c before collapse	2.07	1.87	4.00	5.86

k) Non-conforming horizontal longitudinal displacement of tunnel face (u_f)

It is reckoned that 3D finite element analyses are still too time consuming for routine use in practice (Negro, 2009). Two dimensional analyses are yet preferred in routine projects with account for 3D effects via ground stress reduction factors. If results of 3D numerical modelling are not available, approximate 3D numerically derived solutions may be of some help. Parametric 3D linear elastic analyses for full face shallow tunnels provided by Negro (1988), for lined and unlined circular tunnels, with cover to diameter ratio ranging from 1.6 to 2.8, and K_o from 0.6 to 0.9 and with an increasing elastic soil modulus with depth, furnished an estimate of the maximum horizontal tunnel face extrusion $\boldsymbol{u_f}$ given by:

$$u_f \approx 0.5\, D\, \sigma_{cro} / E_{so} \qquad (10)$$

where $\boldsymbol{\sigma_{cro}}$ is the in situ radial stress at tunnel crown and E_{so} is the in situ tangent modulus of elasticity at springline elevation.

Face extrusion can be measured by longitudinal multipoint extensometers in open face tunnelling. In closed face tunnels, it can be measured by vertical inclinometers at the tunnel axis. If the measured extrusion is larger than *one* to *two* times $\boldsymbol{u_f}$, the ground behaviour ahead the face is likely non elastic or a *non-conforming* condition is present and requires investigation. This is not actually a limiting displacement condition but just an indicator of some sort of non-conformity (of ground parameters or of the estimate proper).

Table 18 presents measured and calculated horizontal displacements of the face at some tunnel projects. Non-conformities were not registered during face passage in these projects. It should be noted that while the calculated movements are right at the face plane, the measurements were taken at some distance behind as indicated in the table. Thus, estimated values were expected to exceed the observed movements (hence measured to estimated movement ratio less than one could be expected, as shown).

Table 18. Observed and calculated maximum horizontal movements near the face of some tunnel projects (from Negro, 1988).

Tunnel	D (cm)	H (cm)	Soil	E_{so} (MPa)	Dist. from face to meas.pt. (m)	Measured movement (mm)	Estimated movement (mm)	Measured to estimated face movement
Victoria Line (anchor A)	4.11	24.0	London clay	60	0	18	24	0.75
Washington C-Line, 1st (SI17)	6.45	11.7	Sands	35	1.5	6	22	0.3
Butterberg - Osterode	11.5	14.6	Sandy gravel	150	1.3	14	13	1.1
Edmonton LRT-sth (SI12)	6.17	9.8	Stiff till	70	3.6	4	7	0.6

The limitations in using this criterion for assessing non-conformities at tunnel face passage are clear: the necessity to know the in situ ground tangent modulus of elasticity, the difficulty in measuring the maximum face extrusion right at the face, the possible non-linear ground response ahead the face in soft soils not expressing a face collapse condition, the approximate nature of the extrusion estimate proper.

l) Measured to calculated crown settlement ratio at tunnel face.

If results of 3D numerical modelling are not available, the calculated dimensionless crown settlement as defined in (j) above, at the face vertical plan was given by Negro et al (1986) as:

$$U_{cf} = 0.375\text{-}0.147K_o \quad (11)$$

This is derived from the same parametric 3D finite elements analysis referred to in section (k) above. Therefore the crown settlement can be estimated by:

$$S_{cf} = D\,\sigma_{cro}\,(0.375\text{-}0.147K_o)/E_{co} \quad (12)$$

The crown settlement can be measured by deep settlement points or by vertical extensometers. The accumulated frequency distribution of measured to calculated crown settlement ratios at the face in more than 50 tunnel projects are presented in Figure 85. Tunnel references are given by Negro (1988). An adjustment in the measurements was performed as they are normally taken at a certain distance above tunnel crown. Accordingly, extrapolated measurements were considered at tunnel contour. After a Kolmogorov-Smirnov adherence test, it was found that a log normal accumulated frequency distribution fits better the data than a normal Gaussian distribution and it is displayed together with the ordered accumulated frequency of the case histories in Figure 85. It is noted that in 50% of the cases, the settlement ratio is close to 1.0 and that in 90% of the cases this ratio varies between ***0.3*** and ***2.0***. The cases in which this ratio exceeded ***2.0*** (10% of the cases) were identified as having shown poor ground control and excessive loss of ground through the face. In other words, a non conforming condition is at play for ratios greater than ***2.0***. This figure could be taken as a ***safe ultimate limiting*** value.

m) Measured to calculated transverse springline displacement ratio at tunnel face.

Similarly to the crown settlement at tunnel face, the springline transverse displacement at tunnel face can also be assessed by the 3D finite element derived results provided by Negro et al (op. cit.), that furnished the dimensionless horizontal displacement of the springline by:

$$U_{sf} = 0.210 - 0.033/K_o \quad (13)$$

From it one obtains the springline transverse displacement at the vertical section containing the tunnel face as:

$$S_{sf} = (0.210 - 0.033/K_o)\,D\,\sigma_{sro} / E_{so} \quad (14)$$

in which $\boldsymbol{E_{so}}$ is the in situ ground tangent elastic modulus at springline elevation and $\boldsymbol{\sigma_{sro}}$ is the radial in situ stress at springline

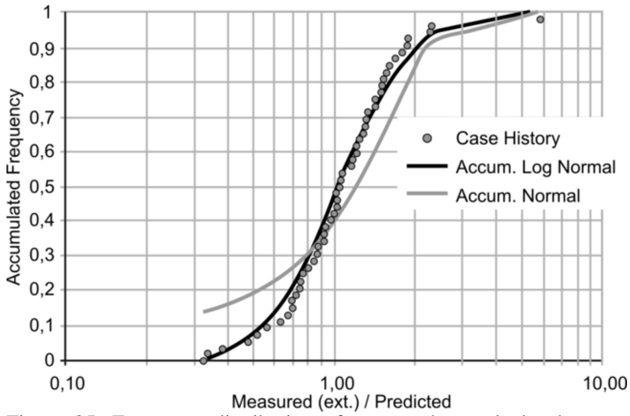


Figure 85. Frequency distribution of measured to calculated crown settlement ratio at tunnel face of some tunnels.

The horizontal displacement of the ground mass at springline elevation is usually measured by means of slope indicators with groove A direction installed normal to the tunnel. The accumulated frequency distribution of measured to calculated transverse springline displacement ratios at the face in more than 20 tunnel projects are presented in Figure 86. Projects references are given by Negro (1988). Adjustments in the measurement were performed as they are normally taken at a certain distance aside tunnel wall. Accordingly, extrapolated measurements were considered at tunnel contour. After a Kolmogorov-Smirnov adherence test, it was found that a log normal accumulated frequency distribution fitted better the data than a normal Gaussian distribution and it is displayed together with the ordered accumulated frequency of case histories in Figure 86. It is noted that in 50% of the cases, the displacement ratio is less to one and that in 90% of the cases this ratio varies between ***0.15*** and ***2.0***. Considering the reduced accuracy of movements measured with inclinometers as compared to precise levelling, a larger scatter of data is anticipated and in fact noted in Figure 86. Hence, it is liberally assumed that a non conforming condition is at play for ratios greater than ***2.0***, being not clear how safe this assumption can be for an ultimate limiting displacement ratio.

n) Measured to calculated longitudinal distortion ratio.

For a stable tunnel face condition (factor of safety greater than 2) and if results of 3D numerical analysis are not available, the maximum longitudinal distortion measured with deep settlement points at a distance 0.3D above the crown γ_{dmax} (typical for settlement points installed above tunnels) can be estimated from results of 3D parametric finite element analysis by Negro (op. cit.) that furnished the expression:

$$\gamma_{dmax} \simeq 0.6\,\sigma_{cro} / E_{co} \quad (15)$$

where the symbols have the meanings already explained. It occurs at points located D/12 behind tunnel face and was derived for tunnels with cover to diameter ratio from 1 to 3, K_o

between 0.5 and 1, and diameters between 4 and 6m. A *non-conforming* condition is suspected to occur if the ratio measured to calculated distortions at this point were greater than ***1.0***, this being understood as a limiting serviceability condition.

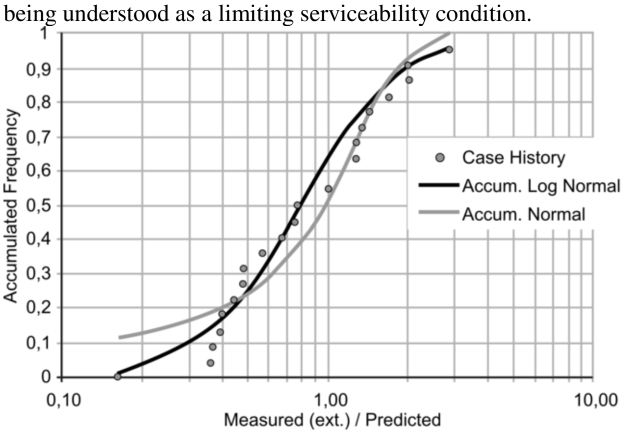


Figure 86. Frequency distribution of measured to calculated transverse springline displacement ratio at tunnel face of some tunnels.

o) Limiting dimensionless crown settlement increment after prefabricated lining installation.

Tunnels lined with prefabricated linings (pre-cast concrete, steel and cast iron segmented rings, steel-ribs and wooden lagging) require some overcutting for segmented liner assemblage. If overcutting is excessive, if voids are left undetected and un-grouted behind the lining, or else, if lining expansion is limited or inefficient, ground collapse may occur behind the liner.

The dimensionless crown settlement at the section where the prefabricated lining is put into contact with the soil, after leaving the shield tail at a distance X measured from the tunnel face, can be estimated by the approximated 3D numerically derived solution presented by Negro and Eisenstein (1997):

$$U_c = a - b.K_o \tag{16}$$

in which the coefficients ***a*** and ***b*** are given by Figure 87.

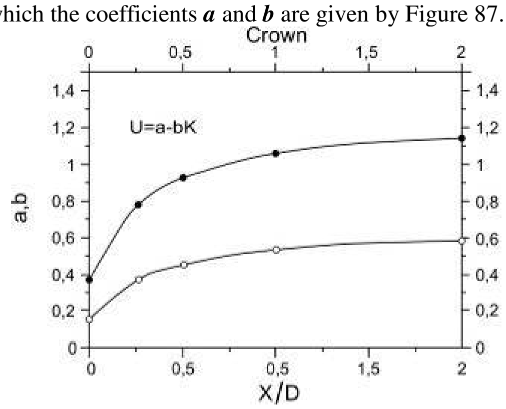


Figure 87. Coefficients of dimensionless crown settlement as a function of the distance to the face.

It has been shown (item *j* above) that soil collapse is imminent when the dimensionless crown settlement is greater than 1.8. Therefore, a near ***ultimate limiting*** increment of dimensionless crown settlement after lining installation is given by:

$$\Delta\, U_{cult} = 1.8 - U_c = 1.8 - (a - bK_o) \tag{17}$$

For K_o equal to 0.5 and for the lining activation at a typical distance from the face of one diameter, one gets:

$$\Delta\, U_{cult} = 1.8 - 1.059 + 0.537x0.5 \simeq 1.0 \tag{18}$$

Figure 88 shows the distributions of increments of crown settlement $\boldsymbol{\Delta U_c}$ taking place after lining was installed in more than 40 tunnel projects (Negro, op. cit.). These were calculated from the increment of crown settlement measured after support was installed, until short term movements stabilized. Since they were measured at a certain distance above crown, measurements were extrapolated to the tunnel contour. The distribution of $\boldsymbol{\Delta U_c}$ was separated into three groups of support: a) sprayed concrete, b) prefabricated and grouted linings, c) prefabricated linings expanded against the soil. Main difference between support groups refers to the 'quality' of lining-ground contact. For most sprayed concrete linings it is reasonable to assume that a full lining-ground contact may exist. In the other two groups this may not be the case: crown displacements may not be solely related to the relative stiffness of the support, but are, to a great degree, also affected by the size of the void left behind the lining. As shown in Figure 88, in more than 70% of cases of sprayed concrete linings, or even more than that if cases of non circular profile were excluded, $\boldsymbol{\Delta\, U_c}$ is equal or less than ***0.3***. If this figure is taken as a reference value for the limiting crown settlement increment of a tunnel with good support-soil contact, one would say that for prefabricated and grouted supports only 40% of the cases responded as having such a contact quality. A slightly higher accumulated frequency (56.5%) is noted for prefabricated and expanded supports, indicating a marginally better ground control condition than that of grouted systems. Accordingly, one notes in Figure 88 that in a considerable number of cases of prefabricated linings, ultimate crown settlement increment may have been exceeded and ground might have reached near collapse condition though tunnels have not in fact collapsed as they were already lined. This was the case of reported ground collapse behind precast liners in Sao Paulo, Edmonton (Negro, op.cit.) and in O Porto (Barbendererde, Hoek, Marino and Silva Cardoso, 2006).

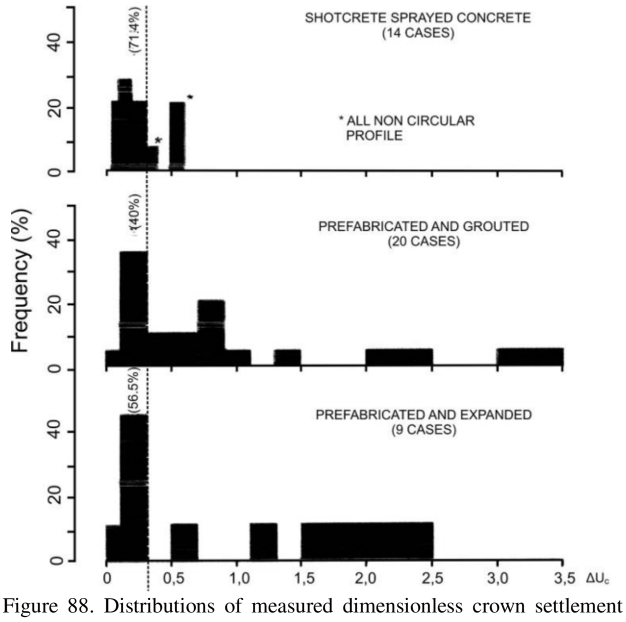


Figure 88. Distributions of measured dimensionless crown settlement after lining installation.

p) Discussion on performance indicators.

It might be tempting to define performance parameters related to the maximum or to the minimum total pressure measured on the cutting face of a TBM, based on ultimate face pressures provided by solutions of the Plasticity Theory. If the shallow tunnel face is approximated by a spherical cavity, the limiting collapse pressure of a fluid inside the sphere can be provided by Mandel and Halphen (1974) solutions (closed form and numerically derived). However this may not be so simple for EPB with foam added to the soil mixture. Bezuijen et al (2005) found in the Botlek Tunnel that the effective stress of the muck in the pressure chamber can be virtually equal to zero and that the total pressure is highly dependent on the excavation process (sign of cutting face rotation, stoppage frequency and duration, etc). For slurry shields, conditions tend to be less dependent of the excavation process, but the applied pressure may depend on the TBM cutting rate and on the slurry cake formation rate (outward water flow from face chamber) – see Bakker and Bezuijen (2009).

Table 19 presents a summary of the ***performance indicators*** defined. The table provides the indicator symbol, its definition, concept, acceptable ranges, limiting range, and corresponding limit state. Users of the indicators set could scale the indicators using weights to each indicator and summing up an overall performance mark.

For a large tunnel contract, in which a comprehensive field instrumentation project makes available all input data needed to assess the 15 ***performance indicators*** defined above, the amount of time required to calculate them, with the processed instrumentation data at hand, is not more than two hours. These two hours represent less 1 metre of a sprayed concrete lined tunnel advance or less than 4 metres of a mechanized shielded tunnel advance, both at full speed. These values are much less than the advances observed in the time interval required for reading and processing all instruments involved, which may take half a day of work. In other words, the amount of time to assess the indicators presented do not represent an over increased data analysis and, in fact, add substantial technical and management value to the routine field instrumentation of an urban shallow tunnel. Moreover, it is quite simple to be implemented, requiring minor technical training.

5.5 *Interactive Tunnel Design.*

Interactive geotechnical design is gradually becoming a recognized approach to the solution of certain geotechnical design problems, though legal and risk barriers make it less used in some countries. The Technical Committee 37 of ISSMGE has been formed to address this subject (see http://www.issmge.org/home/page.asp?sid=296&mid=2&Id=401).

Table 20 summarizes how this approach evolved from Terzaghi's original conception early last century, through Peck's attempts to systemize it, to the present understanding of it: "a continuous, managed, integrated process of design (and prediction), construction control, monitoring and review which enables previously defined modifications to be incorporated during and after construction as appropriate" as defined by Nicholson, Tse and Penny (1999) on a research report published in the UK by CIRIA.

This process design is applicable to any geotechnical structure. However, tunnels in soil are ideal structures for its use. This is because soil tunnelling is essentially an industrial process (Negro, 1999), in which the cost of the industrial product is extremely dependent on the production rate of the industrial line. A TBM is easily seen as an industrial line of production as much as a NATM equipped crew is seen as a production team. Ideally, condition for application requires possibility for modification of design during construction, what is not uncommon in most traditional tunnel construction contracts, though less common in TBM contracts. Also ideal is its application *ab initio,* with provision for actions both for most probable and for less favourable circumstances (Peck, 1969). With this set since beginning, optimization of the tunnel construction process is easy to be achieved. Peck set a word of caution in his 1985 lecture when he sensed that his Observational Method was being discredited by misuse, to disguise poor design or "to an excuse for shoddy exploration".

More recent concepts incorporated into this approach are the use of robust instruments, monitoring redundancy and, the *progressive modification* approach in which construction is started with an acceptable level of risk and is modified accordingly (Powderham and Nicholson, 1996). More importantly is the current perception that the observational method works best where ductile failure mechanisms occur. The differences between ductile and brittle scenarios were summarized by Nicholson (1996). Table 21 presents comparisons between these scenarios for tunnels in soil: it is largely based and adapted from Nicholson (op. cit.). One should be particularly careful when dealing with a ductile ground environment but using a brittle support or lining system: they are incompatible and should not be used for the best use of the interactive design. On the other hand, with a brittle ground mass, avoid as well the use of brittle support or lining system, for the sake of safety. These two conditions led to the decline of unreinforced concrete lining in Brazil, where minimum steel reinforcement are usually specified in the design of the tunnel lining (see Negro, 1994 and Negro, 2006).

Table 19. Summary of Performance Indicators for shallow tunnels in soil.

Item	Symbol	Definition or Concept	Limiting Values	
			Serviceability	Ultimate State
a	S_c/D	Limiting crown settlement to tunnel diameter ratio.	0.03 to 0.04 (or minimum required clearance)	0.03 to 0.15
b	S_s/S_c	Limiting surface to crown settlement ratio at tunnel axis (or settlement increments ratio).	-	1.0
c	γ	Limiting subsurface distortions.	-	1/10 for soft and 1/30 for stiffer soils
d	LDI	Longitudinal distortion index.	-	Change in sign
	θ	Displacement vector orientation.	-	Increasing angle to vertical
e	$\%V_s$	Volume of surface settlement.	0.5 to 1%	3 to 40%
f	$\%V_l$	Volume of soil lost (loss of ground).	0.5 to 4% (usual.) 4 to 6% (excep.)	8 to 10%
g	$\Delta D/D\%$	Lining distortions.	0.05 to 0.75% (dep. on soil)	-
h	%Overburden	Reference lining load.	25 to 75%	-
i	p	Maximum lining load.	-	$3EI / R^3 + E_s(1+\nu)$
j	U_c	Dimensionless crown displacement.	1.0	1.8
k	u_f ratio	measured to calculated maximum face extrusion (u_f) ratio.	> 2 (non-conforming)	-
l	S_{cf} ratio	Measured to calculated crown settlement ratio at tunnel face (S_{cf}).	1.0 (non-conforming)	2.0 (safe)
m	S_{sf} ratio	Measured to calculated transverse springline displacement ratio at tunnel face (S_{sf}).	1.0 (non-conforming)	2.0
n	γ_{dmax} ratio	Measured to calculated longitudinal distortion ratio (γ_{dmax}).	> 1.0 (non-conforming)	-
o	ΔU_{cult}	Limiting dimensionless crown settlement increment after prefabricated lining installation (ΔU_{cult}).	0.3 (safe)	1.0

Table 20. The development of the interactive geotechnical design.

Year	Author	References	Method	Features
1945	K. Terzaghi	Peck (1969) Bjerrum et al (1960) Terzaghi (1961)	Experimental method, learn-as-you-go method, design-as-you-go	Inventory of differences between reality and assumptions. Based on the latter, compute quantities to be measured. Close gaps in knowledge with measurements, modify design if needed.
1969	R. B. Peck	Peck (1969)	Observational method	Two types of applications: best way out of difficulties and (ideal) *ab initio* applications.
1985	R. B. Peck	Peck (1985)	Observation method: a word of caution	The role of finite-element methods with the OM. OM becoming discredited by misuse. OM requires thorough investigations and definitions of course of actions for most probable circunstances and for less favourable. OM not to disguise poor design or to excuse for shoddy exploration.
1996	A. J. Powderham and D. P. Nicholson	Powderham and Nicholson (1996)	The way forward	Contractual constraints to OM. Monitoring redundancy is required. The "progressive modification" approach to OM. Importance to identify ductile and britlle behaviour. OM as a "process design". Need of risk assessment.

Table 21. Ductile and brittle scenarios for tunnels in soil and the impact on interactive design (modified from Nicholson, 1996).

Feature	Ductile	Brittle
Failure development	Gradual development of ground settlements with large ground losses, intense lining cracking and large distortions.	Abrupt failure, after limited ground settlement, small loss of ground, minor lining cracking and smaller distortions.
Governing limit state	Serviceability.	Ultimate.
Predicability based on experience	Reasonable, rich, case histories.	Limited, less case histories.
Numerical predicability	Reasonable predictions.	Difficult and complex due to strain softening.
Instrumentation	Simple instrumentation is valuable.	Simple instrumentation may not detect pre-failure displacements.
Contingencies	Fairly ample time for action.	Too short time for proper action.
Impact on interactive design	Good, induced damages can be controlled, requires use of ductile lining for conformance, good conditions for optimizations and savings.	Bad, requires conservative design and construction, requires use of ductile lining for safety, poor opportunities for optimizations and savings.

5.6 *Selected Case Histories.*

For reading reference, two classes of tunnel case histories were selected: a) Research Oriented Monitoring and b) Large Routine Projects. The first refers to investigations on the fundamentals of certain tunnelling performance aspects. Two cases of large tunnels in soft ground of this class are referenced, both built in the Netherlands: the Botlek Rail Tunnel, driven with an Earth Pressure Balance tunnelling machine and the 2nd Heinenoord Tunnel, driven with a slurry shield machine.

In the Botlek Rail Tunnel two performance aspects that were investigated are highlighted. First, the total and pore water pressure changes measured in the EPB machine pressure chamber, in which sand was conditioned with foam. A somewhat surprising fact was found: near zero effective stress in the sand-foam mixture (Bezuijen et al. 2005a, Bezuijen et al. 2005b and Bezuijen et al. 2006a). It appears that it is still to be explained the mechanics of the effective stress balancing the ground mass ahead an EPB with the foam conditioned sand in the pressure chamber. Second, the lining loads induced by backfilling grouting. An also surprising result came up from measurements of fresh grout pressure gradients inducing lining buoyancy in the grout, not followed by the heavier TBM, thus generating longitudinal bending of lining and concentrating loads on the TBM tail (Bezuijen et. al., 2006a and Bezuijen et. al. 2006b). Buoyancy of expanded segmented lining was known to take place in short-length TBMs but the role of fresh grouting in enhancing it only now was made clear.

In the 2nd Heinenoord Tunnel also two performance aspects investigated are highlighted. First, the importance to take into account the effects of the backfilling grout on the soil displacements (van Jarsveld et al. 2006), an eluding aspect of some design specifications, failing to reckon it as an important tool designers have at hand to control loss of ground and settlements. Second, the excess pore water changes measured in sand in situ, ahead of the slurry shield face, during TBM drilling. Bezuijen et al. (2006c) and Bezuijem and Talmon (2009) measured pronounced increase of pore pressure ahead the TBM cutting face, due to the removal of the bentonite cake by the cutting tools, inducing a transient ground water flow away from the face. The potential consequences of this finding is yet to be completely explored.

The second class of tunnel case histories, large routine projects, refers to monitoring conducted in major soil tunnel projects in urban areas, that added considerable contribution to engineering practice through detailed monitoring and investigation. Also two cases of large tunnels were selected, both in clayey soils: Toulouse Subway and Brasilia Metro. The first refer to TBM driven tunnels whereas in the second, tunnels were driven by traditional tunnelling method, using sprayed concrete as primary and secondary lining.

For Toulouse Subway, a rather involving field investigation program assessed the comparative performance of three tunnel construction technologies in molasses, gathering important findings regarding the response of a particular ground to distinct construction techniques offering distinct ground control conditions (Emerault, et al. 2005). Details and basic data of this project is available at the TC28 Data Base: http://tc28.insa-lyon.fr/

In the South Wing Brasilia Metro, the project was developed by an ab initio Observational Method with tunnels driven through a peculiar tropical soft clay, with a very porous structure, that some contended as collapsible but in fact is simply a higly contracting structured soil, showing large volumetric changes upon shearing. This was an application of Interactive Design in a very ductile environment that used the performance indicators presented earlier for assessment of response conformity (Negro, 1998a and Marques, et. al., 2006). A number of construction sequences were designed for a tunnel with increasing degree of ground control and cost, and were used accordingly as planned or as required by the observed performance.

In one section, tunnel failure was precluded by an *ad hoc* design change, triggered by the performance indicators analysis. Non-conformity was later found to be caused by an unapproved installation of geomembrane in a concrete lining construction joint.

6. GEOTECHNICAL INSTRUMENTATION

6.1 *Foreword*

It is well known that geotechnical engineering is by no means an exact science and that every project required taking account of earth and rock conditions run the risk of surprises. It has long since been advocated that field observations coupled with high quality measurements provide a viable means of obtaining applicable design information, checking of design conditions, verifying the validity of design during construction, enabling changes to be made in accordance with predetermined trigger systems, and verifying anticipated effects of new construction or changing conditions on existing structures (e.g. structural health monitoring). Geotechnical monitoring with specifically designed instrumentation only truly came to the front in the 1940's to 1950's. At the time, geotechnical engineering itself was in its infancy and geotechnical monitoring was driven and operated by dedicated engineers using mostly simple mechanical and hydraulic instrumentation (Dunnicliff, 1988). By 1988 Dunnicliff was of the opinion that although significant advances were made in instrumentation coupled with advances in technology, the wider use of geotechnical instrumentation attracted people in the geotechnical industry who were not themselves dedicated to spend time to learn the instrumentation. This was detrimental to the development of the instrumentation industry and the successful use of geotechnical instrumentation in general; as such Dunnicliff (op. cit.) focused his contribution on proper planning of monitoring systems. Twenty years later, Dunnicliff's general principles remain valid even though technologies have progressed. In practice, a significant amount of monitoring remains at the mercy of budget-driven processes and the less than optimal utilisation of available monitoring technologies to further the state of the art in geotechnical engineering. With this as background, this section of the report aims to:

- review the basic principles and requirements of geotechnical instrumentation (and the people driving geotechnical monitoring);
- highlight some new advances in geotechnical monitoring, especially as pertaining to soil investigation, high capacity pile load testing, and fibre optic sensing and;
- provide case studies to illustrate the recent successful use of novel instrumentation technologies.

6.2 *Review of Requirements for Instrumentation Planning and Selection of Instruments*

6.2.1 *Basic benefits of geotechnical instrumentation*

The general benefits of geotechnical design as listed by Dunnicliff (1988) remain valid and are reviewed here (albeit with more recent examples) for the purpose focussing the reader in terms of his own possible application.

Benefits During Design

From a design point of view, geotechnical instrumentation is frequently used for many purposes:

- Define soil conditions. This encapsulates the widely applied monitoring of groundwater conditions, in situ soil stresses (very difficult to measure) and deformability, or defining ground and rock profiles in terms of cavernous and karstic conditions in difficult dolomite and limestone sites.
- Proof testing of foundation systems that require novel design or difficult ground conditions where design parameters are difficult to predict based on existing databases. This is especially of significance nowadays in places such as the UAE and China where structures are being designed that extend beyond anything designed and constructed ever before.
- Validating design calculations, such as sophisticated finite element or finite difference calculations where the outcomes of the calculation are highly reliant on particular parameters.
- Fact-finding in a crisis, such as recent collapses of the Nicoll Highway tunnel in Singapore in 2004 and the Infinity Tower basement excavation in Dubai in 2007.

Benefits During Construction

During construction, the benefits of monitoring can be utilized primarily for:

- monitoring for safety, e.g. deflection monitoring during excavations;
- observational method, whereby monitoring is used to indicate the appropriate use of predefined courses of action depending the correspondence or deviation of responses from anticipated behaviour in design.
- construction control;
- legal protection, e.g. monitoring of existing infrastructure that may be affected with current construction activities;
- measuring of quantities, e.g. embankment settlement to estimate fill quantities;
- enhancing public safety where by monitoring is used (and advocated) to monitor effects that may impact on public safety, e.g. monitoring during the launching of a bridge across a busy road; and
- advancing the state of the art by allowing research teams to monitor specific issues, conditions, elements of construction that would enable new data to be available to conduct research to solve a problem at hand.

Benefits after Construction

Long-term monitoring of structures are advocated more frequently in recent times to ensure structural health from changing conditions in the immediate environment of a structure (termed structural health monitoring). Structural health monitoring is a major field of research and impacts on various industries of which civil engineering (and geotechnical engineering) is but a small portion.

6.2.2 *General Principles of Monitoring*

Although the readers of this report will generally be experienced geotechnical professionals with at least some instrumentation experience, it is worthwhile to reiterate some of the well-known principles of monitoring to set the stage for the existing advances in sophisticated monitoring made recently. Dunnicliff (1988) remains one of the most authoritative sources for engineers proposing to select and design suitable geotechnical monitoring systems. The following general principles are stated and remain valid for any practitioner embarking on a monitoring project.

Instrumentation and Data

- Instrumentation cannot guarantee good design or construction without any problems.
- Instrumentation should be selected and positioned to obtain a specific answer that can be interpreted and related to answer the issue at hand.
- Instrumentation should only be utilised to monitor behaviour that is difficult to know beforehand; i.e. do not monitor what is obvious to see.
- Good information regarding possible influence on the structure or element being monitored must be gathered throughout the monitoring process; for instance construction records, weather during monitoring and other prudent factors.

- Data handling (acquisition and processing) should fit the application; very sophisticated data acquisition and very high data rates may not be necessary for slow actions and could waste scarce calculation and interpretation resources.
- Suitable sensitivity and reliability are the most important requirements for instrumentation. The sophistication of the instrument should be chosen specific to the application, which means that high technology instrumentation is not necessarily the most suitable way forward.
- Instrumentation hardware and the choice of monitoring installation should not be chosen primarily on cost.
- Robustness and redundancy of the instrumentation system are essential requirements to ensure a successful monitoring system. There must be sufficient number of monitoring points to allow for inevitable instrumentation failure or damage. Sufficient number of instruments is also required to enable capturing the intricacies of ground variations and/or structural element behaviour to ensure that a meaningful and coherent picture of the data is possible.

People

- The installer must both have a background in the fundamentals of geotechnics and the intricacies of the instrumentation to be used to be able to anticipate the requirements for installation (e.g. stress relief, desaturation of a hydraulic instrument, etc.) and possible unexpected monitoring responses during commissioning.
- Successful installation relies on dedication of the installation team and their commitment to work under conditions (e.g. late at night) which would improve the success of the installation.
- The engineer who will interpret the data needs to know the intricacies of the instrumentation and needs to be able to anticipate the reaction of an instrument in the application it is used. In addition, the engineer also needs to understand whether deviations from expected instrument responses could be remedied by calibration or corrections or whether (under the specific circumstances) no valid result is possible.
- Instrumentation goes hand-in-hand with interpretation strategy and reaction protocol. These must be planned and defined prior to commencement of monitoring in relation to the purpose of the instrumentation.

Systematic Approach to Monitoring Programmes using Geotechnical Instrumentation

Planning a monitoring programme is similar to any other component of engineering design and needs to follow a series of logical steps, starting from the definition of the project and ending with a formal drawing and specifications for monitoring. Table 22 provides a basis to systematically plan a suitable monitoring programme (Dunnicliff, 1988; Glišić and Inaudi, 2007):

Table 22. Systematically planning a monitoring system (based on Dunnicliff, 1988).

Approach	Description
1. Define the project conditions	Project layout and geometry, subsurface stratigraphy, engineering properties of the soil, groundwater conditions, nearby infrastructure, environmental conditions, planned construction method
2. Predict mechanisms that control behaviour	This step entails having an understanding (or at least having a working hypothesis) of how the system to be monitored would behave to be able to identify critical aspects of behaviour to be monitored.
3. Define the specific questions that need to be answered	Instrumentation and monitoring programme need to be able to answer specific questions. These questions form the basis of choosing the correct type of instrumentation.
4. Define the purpose of the instrumentation	The purpose of instrumentation must be clear. If the purpose cannot be clearly defined and there is any risk that the monitoring results would not be used the monitoring programme should rather be aborted.
5. Select parameters to be monitored.	Based on (3) and (4) parameters can be defined. It is often required whether the cause or effect (or both) should be monitored. In conjunction this aspect of planning will indicate whether point measurement or continuous measurement is required in relation to the aspects that affect a particular parameter (e.g. pressure may be affected by local geological, moisture, anomalies and other effects, which may necessitate the use of a number of measurements rather than a single measurement)
6. Predict magnitudes of changes	Choosing the appropriate measurement ranges and sensitivities can only be done when the magnitudes of changes to be monitored are known. Predicting magnitudes of change is also used to define appropriate trigger levels for interpretation and action in relation to measurements made. Any good monitoring system must include an effort to predict the anticipated behaviour to be monitored.
7. Devise remedial action	When for instance used for construction purposes it is necessary to have a *predefined* set of conditions and related actions whereby monitoring data could be interpreted on site.
8. Assigning tasks for design, construction and operation phases	A basic principle is that the party with the greatest stake in the data should be given direct line responsibility for producing the data accurately. Dunnicliff (1988) proposes a typical set of tasks and associated responsibilities of the Owner, the Design Consultant, the instrumentation specialist and the Construction Contractor. This predetermination of responsibilities provides a clear assignment of tasks and places the desired focus on each of the interested parties in terms of procurement, installation, interpretation and implementation responsibilities in terms of agreed contractual responsibilities.
9. Appropriate selection of instrumentation	– The overriding factor of any monitoring system must always be reliability. Instruments must have proven past performance records and must have the best chance of remaining durable and reliable under the particular conditions that it will be used; this includes both installation and final use conditions. It is therefore essential that the likely influences on the instrumentation be listed to enable a proper decision to be taken in regard to appropriate instrumentation (e.g. large deformations, shock, impact, corrosiveness, high pressures even if only temporary, temperature extremes, vandalism, dust, dirt, rain, erratic power supply, loss of access to instruments, etc.). – Following closely onto the requirement for reliability is the requirement that the *installer* and the *engineer* must be familiar with the instrumentation chosen. – Whether the instrument needs to be calibrated or corrected in situ. – Anticipating the effect of possible malfunctions and the possible effect on the remainder of the system. – The technology used must be reviewed in relation to the Owner's expectation of the instrument; e.g. with new technologies in a research phase it must be anticipated that the system might fail and a suitable back-up must be in place.
10. Selecting instrument locations	Choose instrument locations in positions where direct comparison can be made with design parameters or expectation (e.g. put an instrument to measure deflection at the position of maximum deflection predicted in the design). This would also make back-calculation of the 'true' measured behaviour easier as it would be directly compatible with what was calculated in design. These are primary instrument locations. Sufficient redundancy should be allowed in terms of other locations to anticipate that the predicted locations were possibly incorrect (i.e. anticipating that the design models did not predict the 'true' mechanisms of behaviour entirely correctly). Such locations are referred to as secondary instrumentation locations. These locations will also allow comparisons and cross checks of behaviour.

Table 22. Systematically planning a monitoring system (based on Dunnicliff, 1988). (continued).

Approach	Description
10. Selecting instrument locations	Location selection should consider the most appropriate locations to ensure survivability of instruments. This includes allowing for sufficient redundancy in numbers of instruments to allow for unanticipated instrument failures or damage.
11. Recording of factors that may influence the monitoring results to enable relating measurements to causes	As a minimum these relate to: – installation details of each instrument (including at least geology; – visual observations of expected and unusual behaviour (including changes in weather conditions); – construction sequence or method used at the time which may influence the reading; – records of subsurface and environmental conditions that might affect readings (e.g. rainfall, sun, shade, etc.)
12. Procedures for ensuring reading correctness	Evidence that the instrument is working correctly must be available. Such evidence may come in different forms and may include: – a duplicate sensing system; – comparing responses with obvious visible structural response of the item being monitored; – considering consistency between responses of different systems (e.g. pore pressure related to settlement in a consolidation environment); – some instruments may have in-place checks.
13. List the purpose of all instruments	This is a cross-check whether all instruments are indeed needed.
14. Plan the budget	This is the initial cost check to ensure that the proposed system is viable cost-wise
15. Compile a specification for instrument procurement	This document enables procurement of the appropriate instrumentation from specialist suppliers.
16. Planning the installation	The installation requires detailed knowledge of the working environment to enable a procedure which would include: – lists of tools and materials; – installation record sheets; – staff training; – planning for accessories (e.g. connection boxes, extension cables, etc.) – user interfaces; – software installation; – installation of sensors and readout units.
17. Planning for regular maintenance and calibration	This would include: – providing for electrical supply; – providing communication lines; – implementation of maintenance plans for different devices; – planning for repairs and replacements; – providing for storage of maintenance items.
18. Data management	Written procedures should be prepared and would typically address: – execution of measurements; – storage of data; – providing for access to data; – visualization of data; – export of data; – data interpretation; – data analysis; – data usage.
19. Closing activities	Planning for closing activities include having written procedures for: – interruption of monitoring; – dismantling of the monitoring system and; – storage of monitoring components.
20. Finalise contractual arrangements for field instrumentation services	All contractual duties shall be made clear.
21. Finalise the budget	The budget includes all professional fees and disbursements.

6.3 *New Advances in Geotechnical Monitoring*

A popular concept that developed across different engineering disciplines is the concept of structural health monitoring (SHM), whereby a view is taken over the lifetime of a project. There is a drive for improving measurement quality, reliability, replacing manual readings and subjective operator judgement, creating easier installation and improving maintenance, whilst also introducing lower cost. This is quite a tall order and any advances on the trusted conventional sensors and instrumentation in any or all of the above requirements may be considered notable advancement of the monitoring environment. This section highlights some advances on three fronts, namely:

- ground investigation;
- high capacity pile load testing and;
- fibre-optic monitoring, especially its impact on spatial and continuous monitoring.

Although not by far exhaustive of all 'new' developments on the instrumentation front, and even though the technologies behind the instruments and sensors to be discussed are not at all new, the following discussion relates to some advances that had a significant and positive impact on the way to monitor in geotechnical industry and the instrumentation available to us.

6.3.1 *Monitoring Geotechnical ground Investigation*

Jean Lutz Parameter Monitoring during Drilling and Grouting

Geotechnical ground investigation is generally relatively slow in its growth, with development frequently focussing on the procedural aspects thereof and defining the amount of data that needs to be achieved for different ground investigation purposes. There has been, nonetheless, over the last couple of years interesting developments on the front of automatized parameter measurement on construction equipment. The objective was to improve the handling of the machine and enabling simultaneous interpretation of ground conditions (or element installation) and construction by reducing the operator judgement of drilling conditions. The Jean-Lutz data recording system (Jean Lutz SA[2]) is one such system that has been developed in France by Dr Jean Lutz and has been fitted successfully to equipment used in installation of bored piles, jet grouting, low pressure grouting, anchor loading, soil mixing, pile driving, vibro flotation, driving of sheet piles, diaphragm wall installation and drilling for ground investigation across the world.

From a ground investigation perspective the Jean Lutz system has recently come into its own for the investigation of karstic dolomite conditions in Pretoria, South Africa for the new Gautrain Rapid Rail Link project. The Gautrain Rapid Rail Link is a state-of-the-art rapid rail network planned for the Gauteng Province. The rail connection is approximately 80km long and comprises links between Pretoria and central Johannesburg, as well as a link between OR Tambo International Airport and Sandton. Between Centurion and Pretoria, in the northern section of the route, the rail alignment traverses about 16 km of dolomitic ground of which 5.8 km will be on viaducts with the remaining portion constructed at ground level.

The challenges of constructing the rapid rail link over the dolomites are numerous, including highly variable ground conditions, presence of voids, erodibility and shallow ground water conditions. If the works are to be completed successfully, it is essential that a thorough understanding of the ground conditions be gained to support detailed technical planning prior to commencing construction. An important part of this understanding relies on good and interpretable drilling information. Some of the challenges about drilling and probing included:

- Extremely difficult drilling conditions in soft (sometimes cavernous) wad residuum interspersed with hard rock chert bands, blocks and gravel and dolomite boulders (referred to as floaters) and highly variable rock head conditions (Figure 89).

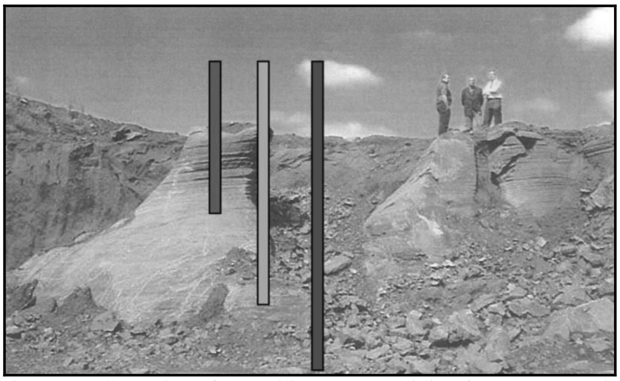


Figure 89. Illustration of how piling (or any drilling for that matter) in the same vicinity in highly variable dolomite rock head conditions can result in vastly different drilling conditions and depth, encountering difficulties with hard rock / soil interfaces, boulders and deep soil conditions over short distances.

  This made for extremely difficult rotary core drilling conditions and very limited sample recovery success, to such an extent that core drilling was abandoned as an appropriate means of investigation in all areas except non-dolomitic ground. Usually, in the dolomites, South African industry relies on relatively crude percussion drilling techniques whereby penetration rate is measured every metre length of drilling using a hand-held stop watch and inaccurate measures for measuring the drilling depth. Information relies heavily on the chip samples recovered and an experienced engineering geologist to obtain a sense of ground profile, water table depth, occurrence of floaters and bedrock variability. The method is applied reasonably successfully for classifying dolomite risk for housing projects, but lacks the necessary precision for robustly providing design parameters for foundation design.

- Conventional probing (e.g. Dutch cone, piezocone, Standard Penetrometer Testing, etc.) and spot stiffness measurements (e.g. pressuremeter) could only be done using onerous pre-drilling to a specified depth, testing of layers which would be soft enough to probe, and continuing drilling to the next test depth. Full-scale load tests indicated that this manner of testing was extremely unreliable and produced unrealistically conservative results.
  A tool was needed whereby the soil mass as a whole could be probed from which a suitably experienced engineering geologist or geotechnical engineer could identify stratigraphy and allocate soil parameters such as stiffness for use in the design of pile, piled raft and raft foundations. Percussion drilling with Jean-Lutz instrumentation, calibrated with full scale load tests (area loads and pile load tests), was used for this purpose. A big advantage of the system was that it could be fitted to existing drilling rigs and the recorded information can be transferred to a PC for processing (or can be edited in 'real time' using an integrated graphic printer). The following parameters were measured and related to soil profiles, in an effort to interpret the soil and rock profile in terms of parameters for compressibility to be used in raft, piled raft and laterally loaded piles:
  - drilling depth;
  - drilling penetration rate;
  - thrust and restraint pressure;
  - water pressure;
  - torque;
  - rotary speed;
  - vibralog (reflected percussion wave) and;
  - flow rates (water, mud and air).

[2] www.jeanlutzsa.fr

An example of Jean Lutz penetration results from four boreholes plotted to indicate harder and softer zones in a dolomite soil and rock profile is shown in Figure 90.

The Jean Lutz system was also very successfully employed for grouting of the dolomite residuum to minimise the risk of dolomite-related subsidence (sinkholes and dolines). It is further used frequently across the world in different grouting projects. In conjunction with the aforementioned drilling monitoring, grout pressure, flow rate and grout volume was monitored using the Jean Lutz system. The Jean Lutz system allowed for automatic control of grout pumps (up to twelve pumps can be operated simultaneously) to terminate grouting at predetermined criteria (maximum pressure and/or maximum grout volume injected). This simplified the usually onerous decision-making process for deciding whether sufficient grout has been placed.

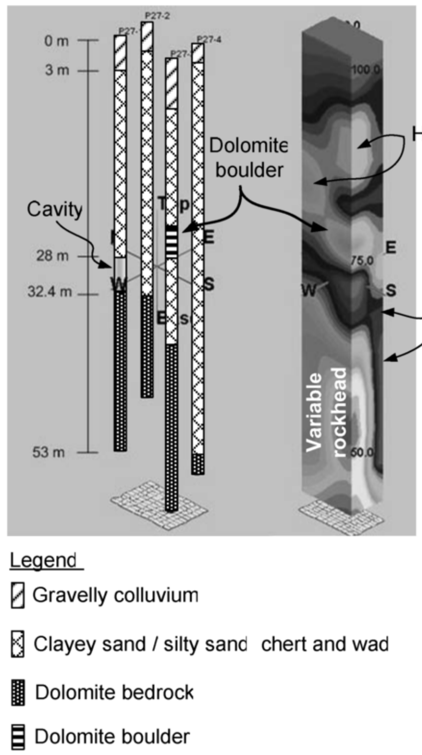


Figure 90. Jean Lutz Penetration data plotted to indicate harder (lighter colours) and softer zones (darker colours) between four boreholes.

Downhole Imaging with Borehole Radar

Borehole radar is traditionally a subsurface detection tool designed for imaging geological targets in resistive formations and a lot of research has been conducted for its use in the mining environment (e.g. Voigt, 2006). It has recently been further developed for use in the civil engineering industry and specifically for subsurface imaging (Van Dongen, 2002). Van Dongen describes the use of borehole radar for detection of the diameter of jet grout columns and the detection of objects and layers during tunnel construction.

An example of the successful use of borehole radar in ground investigation is the Gautrain Rapid Rail Link project. A significant problem with founding end-bearing piles in the dolomite is the steep rock gradient variations and the immediate transition between hard rock dolomite (UCS typically between 50 MPa and 200 MPa) and soft soil conditions, making pile installation extremely challenging. A further problem that arises is the occurrence of voids in the bedrock and the possibility for its occurrence either entirely or partially below the pile toe. During the ground investigation for Gautrain, in areas where piled foundations were envisaged, borehole radar using a 250 MHz antenna/receiver provided a solution to interpret the occurrence of voids and steep rock/soil transitions, using the ground investigation boreholes as installation holes (Storry et al., 2009). Radargram plots were also interpreted to identify reflectors showing geological structure (faults) in addition to the relative degree of signal attenuation, which provided a measure of rock quality. Since borehole radar surveys are omni-directional adjacent boreholes were referenced in the radargram plot through detection in other boreholes in a similar fashion to the example shown in Figure 94. Additional ground investigation was frequently scheduled to investigate anomalies identified in the radargram and on several occasions significant voids were proven by this additional drilling, which may otherwise have been missed by the conventional drilling investigation. Figure 91 shows an example of a circular cavity and a a crack or linear feature in the ground and the corresponding borehole radar response showing both features. The method is a valuable addition to the arsenal of ground investigation instrumentation and addresses the gap between conventional logging measurements in boreholes and seismic surveys (Van Dongen, 2002).

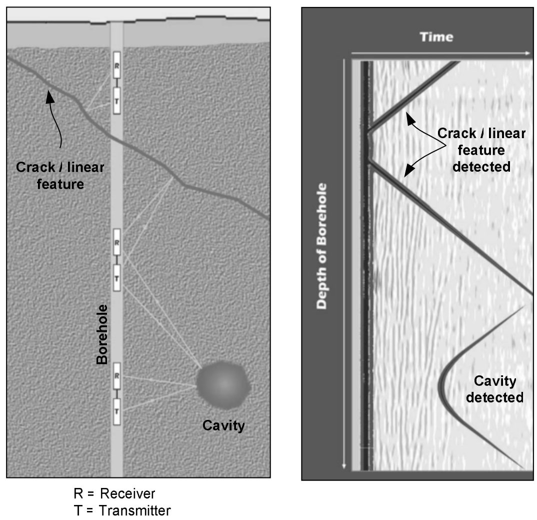


Figure 91. Borehole Radar Output.

6.3.2 *High Capacity Foundation Load Testing*

Pile load testing is frequently used for validating pile resistance and is routinely used with instrumentation such as strain gauges, extensometers, inclinometers and load cells. Difficulties with pile load testing included among others issues such as being able to isolate end bearing and shaft characteristics (or variable shaft resistance characteristics along the length of a pile) and how the resistances contribute to the overall pile resistance. Pile load tests are furthermore often associated with large and bulky setups to create sufficient kentledge or the installation of secondary systems such as ground anchors or anchor piles to derive the necessary resistance for the pile tests. The Osterberg cell® (or O-Cell®) overcame many of these problems (Schmertmann, J., 1993; Osterberg, J. O., 1994; Osterberg, 1998). The O-Cell® is a hydraulically driven, high capacity, sacrificial loading device installed within a foundation unit such as a concrete pile. As the load is applied to the O-Cell®, it begins working in two directions, namely upward against upper side shear and downwards against base resistance and lower side shear (if installed higher up in the shaft). By virtue of its installation (Figure 92) within a pile (or foundation member), the O-Cell® derives all reaction from the soil and rock profile; no secondary load systems or bulky kentledge is required. End-bearing provides reaction for the skin friction portion of the test, and skin friction provides reaction for the end-bearing portion of the test. Testing continues until ultimate skin friction capacity is reached, or ultimate end-bearing capacity is reached, or the maximum O-Cell® capacity is reached. Each O-Cell® is instrumented to measure expansion directly. Along with compression and top of pile shaft measurement, this enables the complete identification of downward end-bearing movement and the upward skin friction movement.

What is remarkable is the capacity of the O-Cell®. Individual O-Cells® range from 750 kN (75 tons) to 50 MN (5000 tons) in capacity. By installing multiple O-Cells® at the same horizontal plane up to 300 MN can be achieved (Loadtest, 2005, see www.loadtest2005.net). On the Cooper River Bridge (South Carolina, USA) several load tests were conducted on piles varying between 1.9 m and 2.4 m in diameter, with loads achieving 63 MN on a 67 m deep shaft and 59 MN on a 68 m deep shaft (Ahrens, 2005).

The O-Cell® also provides the opportunity for installing it in different planes to isolate the contribution and behaviour of different sections of a foundation element. It has a further advantage over conventional load testing in the sense that it is relatively quick to install, providing for shorter duration tests. For the Missouri River, Amelia Earhart Memorial Bridge (Ryan, 2005) the second highest load test ever achieved in the USA, was conducted. A multi-level test setup of a 48.7 m long, 1.525 m diameter test pile achieved a combined load of 158 MN. In a world where 'bigger is better' is becoming popular again in civil engineering industry, O-Cell® instrumentation provides a significant way forward in optimising foundation capacity and stretching the frontiers of design.

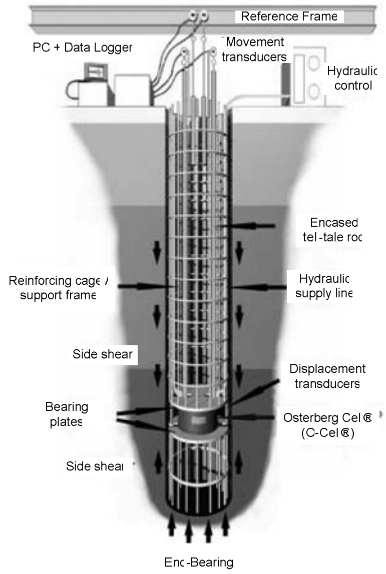


Figure 92. Typical Osterberg Cell® (O-Cell®) monitoring system setup in a pile (courtesy LOADTEST).

6.3.3 *Fibre Optic Sensing*

One of the most exciting advances in geotechnical instrumentation in recent years has been the development of fibre optic (FO) sensors. Where FO sensors have really come into their own is the strategy of life cycle approach when infrastructure owners consider how their investments should be spent over the lifetime of a particular infrastructure in question. In the case of civil structures, reliability and long-term stability is a big advantage of FO sensing (Glišić and Inaudi, 2007). When considering civil structures the following parameters are usually relevant (Glišić and Inaudi, 2007):

- physical - position, deformation, inclination, strain, force, pressure, acceleration, vibration;
- temperature;
- chemical - humidity, pH, chlorine concentration;
- environmental parameters, including air temperature, wind speed, wind direction, solar radiation, precipitation, snow accumulation, water level, flow, pollutant concentration.

These parameters can normally be monitored using conventional sensors. FO sensors are preferred only where it can be shown to be superior in:

- quality of measurement;
- increased reliability;
- where manual readings and operator judgement need to be replaced / minimised using automatic measurement;
- where easier installation and maintenance occurs and;
- if it produces lower cost.

FO also has the following attributes, which makes it very attractive for sensing and supporting the requirements given above for many applications (Glišić and Inaudi, 2007):

- FO does not corrode / erode in harsh chemical environments (i.e. preferred for gas, oil and concrete environments);
- FO is resistant to weathering;
- It is not affected by electromagnetic fields (EM) or electromagnetic interference; it is therefore generally more effective than electrical sensors in an EM, radio frequency or microwave environment;
- FO is explosion proof;
- It has small size in cross-section;
- Measurements can be done over large distances (e.g. large structures, pipelines, multiple bridges along a highway, long slopes, etc.);
- FO can be used for fully distributed sensing, while conventional point sensing cannot practically achieve this without significant cost.

FO Technologies Available in Industry

Although FO technology has been applied in the market for many years, only a couple of technologies really made it to industry as summarized in Figure 93. The versatility of FO sensing is shown in its representation across the range of point sensing, long gauge sensing and distributed sensing.

To relate the differences and intricate properties of each system the measurement strategies of the different FO technologies are described in Table 23 as follows.

Table 24 provides a summary of typical characteristics and performance parameters for each of the FO technologies described.

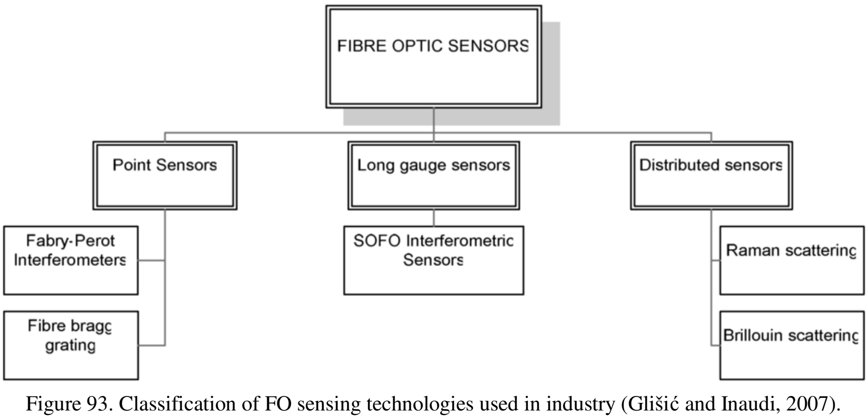


Figure 93. Classification of FO sensing technologies used in industry (Glišić and Inaudi, 2007).

Table 23. Measurement Strategies of popular FO technologies.

<table>
<tr><th>Technology</th><th>Strategy for Measurement</th></tr>
<tr><td>Fabry-Perot Interferometric Sensors</td><td>

– A capillary tube containing two cleaved optical fibres facing each other but leaving an air gap in-between them (Measures, 2001).

– When light is launched into one of the fibres, a back-reflected interference signal is obtained due to the reflection of light on the air-glass interfaces. The interference can be interpreted to identify the changes in the fibre spacing.

– The fibres are attached to the extremities of the capillary tube (typically 10 mm spacing), which means that the gap change corresponds to the average strain variation between the two attached points.

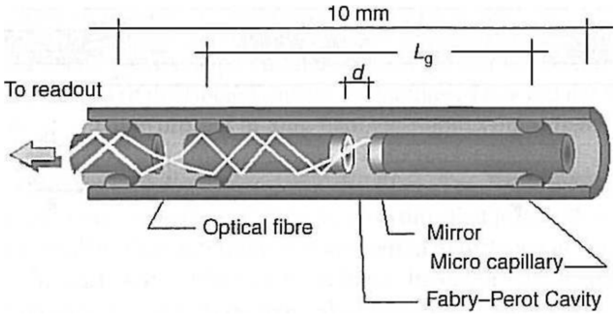


Figure 94. Functional principles of Fabry-Perot sensors (Glišić and Inaudi, 2007).

</td></tr>
<tr><td>SOFO Interferometric Sensor</td><td>

– The SOFO system is a long-gauge fibre-optic deformation sensor with resolution in the micrometer range, known to have very good long-term stability and is insensitive to temperature fluctuation (Glišić and Inaudi, 2007).

– SOFO is an acronym derived from the French for structural monitoring by optical fibres, i.e. surveillance d'ouvrages par fibre optiques.

– The sensor comprises two single mode fibres; one fibre is the measurement fibre and is in direct contact with the structure to be monitored. The second fibre is a reference fibre placed loosely in the sensor housing.

– Deformation of the structure will result in a change of the length difference between the two fibres. The path imbalance created with the two fibres is solved for measurement using a double-interferometer. The first interferometer is made up of the measurement of the two fibres, while the second is contained in a portable unit. The second interferometer is able to introduce a predetermined path imbalance between its two arms.

– If continuous measurement is not a requirement, then the evolution of deformation can be obtained by successive readings. This enables the use of a single reading unit to monitor several different fibre pairs.

– Primarily used for deformation measurement.

– Excellent long-term stability.

– Comprises a pair of single mode fibres of which one fibre (prestressed) is in contact with the structure to be monitored.

– Fibre transmission properties do not affect precision.

– Has been used on bridges, buildings, tunnels, piles, anchored walls, dams, historical monuments, nuclear power plants and laboratory models (Glišić and Inaudi, 2007).

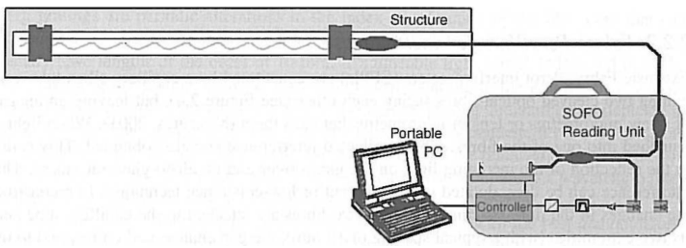


Figure 95. Setup of the SOFO interferometric sensor system (Glišić and Inaudi, 2007).

</td></tr>
<tr><td>Fibre Bragg-Grating Sensors</td><td>

– Bragg gratings are regular alterations in the index of refraction of the fibre core that is produced by exposing the fibre to intense UV light. The gratings typically have lengths of the order of 10 mm. By injecting a tuneable light source into the fibre with gratings, the wavelength created by the grating will be reflected, while the other wavelengths will pass through the gratings undisturbed.

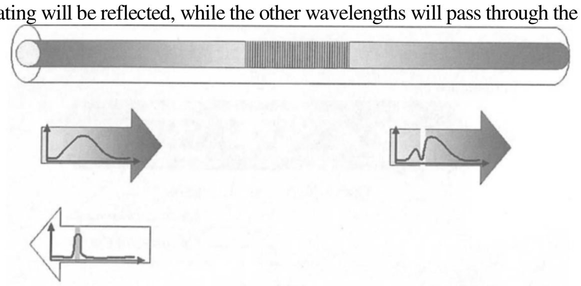


Figure 96. Functional principle of fibre Bragg-grating (FBG) sensors (Glišić and Inaudi, 2007).

– Optical fibre sensors with FBGs are optical strain gauges, with the advantage over conventional strain gauges that they can be multiplexed in the same fibre. This reduces the harness of the installation.

– The gratings are period and temperature dependent. It is therefore possible to measure temperature and strain by analysing the intensity of the reflected light as a function of its wavelength. A tuneable laser with wavelength filter or a spectrometer is used.

– The use of fibre Bragg grating technology for strain and temperature measurement has been used on an experimental and field basis in tunnels, piled foundations, dynamic / vibration measurement for structural health monitoring purposes in the airline industry (Paolozzi and Gasbarri, 2006), corrosion monitoring in concrete (Grattan et al., 2007), aeolian vibrations (Bjerkan et al., 2004).

</td></tr>
</table>

Table 23. Measurement Strategies of popular FO technologies. (continued).

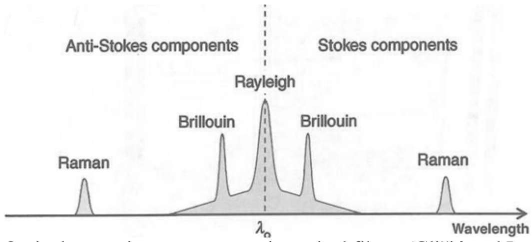
Technology	Strategy for Measurement
Raman-Scattering Sensors	– Optical time-domain reflectometers (OTDRs) have originally been developed for the telecommunications industry and were the start of distributed sensing techniques. – Rayleigh-scattered light is used to measure attenuation profiles of long-haul fibre-optic links. An optical pulse is sent through the fibre and amount of light that is back scattered is measured as the pulse propagates down the fibre. – The detected signal (Rayleigh signature; Figure 100) decays with time and is directly related to the linear attenuation of the fibre. The time information is converted to distance information (if the speed of light is known). – Thermally influenced molecular vibrations cause Raman-scattered light. The back-scattered light therefore carries information on the local temperature where the scattering occurs. – Raman sensing techniques require filtering to distinguish between the temperature-sensitive anti-Stokes component of frequency and the Stokes component (Figure 97). – Raman sensing techniques rely on intensity of light measurement and since the magnitude backscattered Raman light is generally low, the system requires high numerical aperture multimode fibres to maximise the intensity of the backscattered light. – Multimode fibres generally have high attenuation characteristics, which limit the distance range of Raman sensing to 8 km to 10 km. – Long-term accuracy and stability is affected by sensitivity to drifts (Nikles et al., 2004). – Due to its limitation to only measure temperature, it is not widely used nowadays, but was first introduced as a temperature sensor in the 1980's as reported by Dakin et al. (1998). – Like all OTDR systems, the Raman sensing technique is useful for detecting breaks, to evaluate splices and connectors and to assess the overall quality of a fibre link.  Figure 97. Optical scattering components in optical fibres (Glišić and Inaudi, 2007).
Brillouin-Scattering Sensors	– Brillouin scattering occurs because of an interaction between the propagating optical signal and thermally acoustic waves in the GHz range present in the silica fibre. This gives rise to frequency-shifted components. It is observed as the light diffraction on a moving grating generated by an acoustic wave. The diffracted light experiences a Doppler shift since the grating propagates at the acoustic velocity in the fibre (which is directly related to the medium density and depends on strain and temperature). As a result, the so-called Brillouin frequency shift carries information of both local temperature and strain of the fibre (Figure 100). – Whereas Raman-based techniques rely on intensity, Brillouin-based techniques use frequency shift. This makes Brillouin-based systems inherently more accurate and more stable in the long-term. – Brillouin scattering can become a stimulated interaction when an optical signal (probe signal) is used in addition to the original optical signal (called a pump). The interaction causes the coupling between the probe and the pump signals and acoustical waves when the frequency difference between probe and pump light corresponds to the Brillouin frequency shift (a resonance condition is achieved). The resonance condition is temperature and strain dependent and therefore provides a direct measurement of both. – The measuring of the interaction between probe and pump (instead of recording low intensity spontaneously back-scattered light) gives the advantage that the signal-to-noise ration is sufficiently high. This makes for rapid measurement as opposed to long duration required to measure spontaneous back-scattered light. – Due to the use of single mode fibres (inherently low losses occur), Brillouin-based techniques can be used over distances of up to 250 km (Nikles et al., 2005). – Brillouin techniques are applied on a wide variety of industry and research applications ranging from leakage detection (Nikles et al., 2004), pipeline strain monitoring (Vorster, 2005), piling (Klar et al., 2006; Bennet et al., 2006; Sensornet, 2004), tunnel deformation (Mohamad et al., 2007; Mohamad, 2008), mining (Naruse et al., 2007) , dams and lakes (Thévenaz et al., 1998) , railway dynamic effects (Sensornet, 2005), pipeline monitoring (Nikles et al., 2005). – Like all OTDR systems, Brillouin-based sensing techniques are useful for detecting breaks, to evaluate splices and connectors and to assess the overall quality of a fibre link.

Table 24. Summary of FO sensing types and Typical Performance (Glišić and Inaudi, 2007).

	Fabry-Perot Interferometry	Fibre Bragg Grating	SOFO Interferometry	Raman Scattering	Brillouin Scattering
Sensor Type	Point	Point	Long-gauge (integral strain)	Distributed	Distributed
Main measureable parameters	Strain Temperature Pressure	Strain Temperature Acceleration Water level	Deformation Strain Tilt	Temperature	Strain Temperature
Multiplexing	Parallel	In-line and parallel	Parallel	Distributed	Distributed
Measurement points in one line	1	10 - 50	1	10 000	30 000
Typical accuracy	Strain = 1με Δ_1 = 100 μm T_2 = 0.1°C P_3 = 0.25% full scale	Strain = 1με Δ_1 = 1 μm T_2 = 0.1°C	Strain = 1με Δ_1 = 1μm Tilt = 30 μrad	T_2 = 0.1°C	Strain = 20με T_2 = 0.2°C

Table 24. Summary of FO sensing types and Typical Performance (Glišić and Inaudi, 2007) (continued).

	Fabry-Perot Interferometry	Fibre Bragg Grating	SOFO Interferometry	Raman Scattering	Brillouin Scattering
Sensor Type	Point	Point	Long-gauge (integral strain)	Distributed	Distributed
Precision and Stability using the best possible equipment and installation	$?_5$	Strain = 1 με T_2 = 0.1°C	Δ_1 = 2 μm (independent of sensor length, proven over more than 10 years)	T_2 = 0.1°C	Strain = 20με T_2 = 0.2°C
Typical Resolution	Strain = 1με T_2 = 0.1°C	Strain = 1 με T_2 = 0.1°C	Strain = 1 με	Range (length of pulse) = 1 m Strain = 1 με T_2 = 0.2°C	
Range	-	-	20 m gauge	8 km	30 km, 250 km with range extenders; 500 km expected by mirroring 250 km setups, but not yet proven (Nikles et al., 2005)
Fibre type	Multimode	Single mode	Single mode	Multimode	Single mode

Note:
1. Δ = Deformation.
2. T = Temperature.
3. P = Pressure.
4. ? = No reference could be obtained which indicates typical precision.

Case Studies: Fibre-Optic Sensing

This section of the report aims to provide the reader with a sense of the variability and particular strengths of the most widely used fibre optic sensors, namely Fibre-Bragg Grating systems, SOFO systems and distributed sensing using Brillouin optical time domain reflectometry (BOTDR) technology. The case studies are by no means exhaustive but rather provide the reader with particular examples of successful applications of the different fibre optic sensing technologies.

Fibre-Bragg Grating (FBG)

Although fibre optic sensors using FBG have been used successfully in applications where conventional electrical strain gauges are traditionally used, such as bridges (e.g. monitoring cables of a cable bridge by Tian et al., 2004) and tunnels (e.g. rock bolt monitoring by Nellen et al., 2000; temperature and strain monitoring of a tunnel reported by Li et al., 2008), and have been successfully used for temperature detection (e.g. Lönnermark et al., 2008), one of the most distinguishing attributes of optical fibre sensors with FBG is their ability to detect dynamic changes.

A. Dynamic Sensing of Vibration for Structural Health Monitoring (Paolozzi and Gasbarri, 2006)

Paolozzi and Gasbarri (2006) reports that for Structural Health Monitoring (SHM), the occurrence of damage can be associated with the change in vibration characteristics of a structure. The authors report that Caponero et al. (2001) reported the first use of dynamic point strain measurement using embedded FBGs.

The first successful test reported by Paolozzi and Gasbarri (2006) is the successful measurement of local strain measurements using FBGs in a basic experimental setup illustrating its ability to retrieve changes in Strain Frequency Response Functions (SFRFs). This is usually only achieved using accelerometers. The changes in the SFRFs provides the ability to extend measurements from only the total strain integrated over the length of the element (giving only the resonance characteristics), to being able to detect all mode parameters (modal shapes and modal damping) at different positions along the length of a structural element. The authors illustrate this ability by hitting the instrumented alloy bar with a hammer and using Power Spectral Density (PSD) methods to retrieve the first resonant frequency of the cantilevered bar. Although useful as a first indication of structural health, the use of FBGs to obtain mode shapes using by local strain measurement provides a much more promising way forward to detect local damage. Due to limitations of the FBG interrogator to interrogate the signal at higher frequency than the 100.25 Hz resonant frequency of the alloy bar, the authors were only able to detect modal changes at static condition. Further work was planned for the use of a high frequency FBG interrogation system to detect modal changes at higher frequencies as well. Similar good results were obtained using a composite structure.

The authors describe the use of a similar system of FBGs embedded in one wing (1.3 m long) of an unmanned aircraft. The wing was constructed as a sandwich of polystyrene core and one ply of glass fabric. The system was glued together using epoxy resin. The sensors provided 'real-time' strain measurements. The wing was then taken to the laboratory where a controlled test was conducted using both accelerometers and FBGs. Excitation was applied using either an instrumented impact hammer or electromagnetic shaker. Good agreement was obtained between accelerometer and FBG data, although the authors mention that there were limitations in the frequency interrogation characteristics of the experimental setup, disallowing capturing frequencies higher than 20 Hz.

Following the aforementioned experiments, Paolozzi and Gasbarri (2006) affixed FBGs and accelerometers (for comparison) on the Star Tracker (a 7-ton particle detector to be mounted to the international space station). The FBG system was used to satisfy the structural verification plan of the Star Tracker and illustrated the external fixing usage of FBGs (as opposed to embedding during manufacture). The FBGs provided good estimation of strains associated with modal displacement.

The authors conclude that FBGs can be used for performing modal analysis for the purpose of qualification as well as damage assessment. The authors further conclude that, differently from accelerometers, FBGs provide strain information both statically and dynamically without frequency limitation, (the limitation lies in the interrogation system). This they feel is a significant advantage for damage determination using ultrasonic wave propagation.

B. Sensing Reinforcement Corrosion in Civil Engineering Structures (Grattan et al., 2007)

Grattan et al. (2007) reports that the corrosion of steel reinforcement bars in concrete produce waste products that will result in an increase in localised volume of the original steel bar. The localised volume change can be between two and six times the original steel bar volume, which causes a localised strain change. The change in volume could cause cracking and deterioration of the surrounding concrete, as well as loss in cross-sectional area of the load-bearing rebar. Grattan et al. (2007) reports on the response of FBG compared to conventional electrical strain gauges attached to a steel bar embedded in concrete in a laboratory environment to indicate whether it is possible to monitor the localised strain arising from the corrosion process. Two identical blocks were created, one containing a steel bar with several FBG sensors and one with conventional strain gauges. A current was run through the steel bar to effect the corrosion process and the concrete mix was controlled to ensure an environment conducive to corrosion. The objective was to perform the experiment over several months with readings taken every 6 hours on the conventional electrical sensors and daily on the FBG sensors.

The authors report that the FBG sensors demonstrated it could be used for monitoring localised corrosion, distinguishing the strain resulting from corrosion and the strain caused by normal strain situations. The authors did nonetheless report that the FBG sensors provided noisy results. Although not discussed or explained by the authors, similar noisy results due to localised cracking was also observed by Bennet et al. (2006) during curing of the concrete in piled foundations.

C. Measurements of aeolian vibrations (Bjerkan et al., 2004)

Bjerkan et al. (2004) describe the use of fibre optic sensors using FBG technology on a 2910 m long 420 kV overhead transmission line with a diameter of 56.7 mm. The sensors were installed from the bottom of the span to one of the anchoring towers to enable detecting vibrations in the mid span and outside of the end span damping arrangements. The objective was to ascertain whether the system would be successful in evaluating the success of damping systems to ensure that transmission lines in windy areas do not become fatigued. The monitoring programme was conducted over a period of 6 months between November 2002 and May 2003.

Aeolian vibrations can become problematic in flat landscapes with steady winds or fjords and valley areas where high conductor tension is applied. These vibrations are characterised by small amplitudes (typically comparable to the line diameter) associated with frequencies in the range of 3 to 100 Hz, depending on the wind velocity and the conductor diameter. The vibrations may cause fatigue damage to the conductor, and in particular, the suspension clamps, as well as damage to aircraft warning balls attached to the line.

Conventionally the effect of vibrations is measured close to the dampers installed at the end of a span. The downside of this is that vibrations at mid span are not detected. Fibre optic technologies are especially attractive in high electrical power environments due to their immunity to electromagnetic fields, lightweight, small dimensions, explosion safety and the possibility of transmitting signals over long distances using the same cable acting as the sensor also for data transmission.

For the particular application described by Bjerkan et al. (2004), the transmission line spanning a fjord in Norway is tensioned to 385 kN (47% of its breaking load), creating a sagging profile with a maximum sag of 175 m relative to a straight line drawn between two anchor points. The high tension results in very low internal damping in the conductor, making the conductor susceptible to wind-induced vibrations.

Two sensor groups, one located close to the damping system (at chainage 200 m), and one located at the maximum sag position (at chainage 1200 m), were installed onto custom-made slots at 2 m centres created in an aluminium strand. Three sensors were used at each location. Each sensor group was associated to a dedicated fibre optic cable. The aluminium strand was glued to one of the conductor strands and was wrapped along the line with a spinning machine used in telecommunication cables.

The sampling frequency was 100 Hz and the number of measurement points in each recoding was 8500, providing a time series of 85 seconds long. Four series were recorded each hour. To relate measurements to prevailing wind and temperature conditions, wind speed and temperature data recorded every hour from a weather station 2 km from site were obtained.

The authors observe that due to limitations in the availability of measurement equipment the absolute vibration amplitudes were unreliable. They were nonetheless able to distinguish that the strongest vibrations occurred in the maximum sag position, with lower albeit similar distribution of frequencies measured at the position closer to the damping installation. Maximum peak-to-peak amplitudes of 30 mm in the 3 to 5 Hz range were measured in the sag position and concluded that, even though a great number of vibration events were detected, the majority of them were weak and from estimated vibration amplitudes, they concluded that it was unlikely that fatigue damage would occur. The authors concluded that the method based on FBG technology was successful in detecting aeolian vibrations on overhead conductors and can be used to optimize vibration damper arrangements or verify damper performance. Accumulated vibration data could also provide information on the mechanical condition of the conductor and its remaining life. A further advantage identified was that the fibres were able to detect environmental conditions such as ice accretion and temperature monitoring of the conductor.

SOFO

The most marked characteristic of SOFO systems in relation to other systems is its long gauge length, which enables it not to be affected by local effects such as cracking, air pockets, etc., which may occur in concrete structures. The following cases show this attribute being used to great effect.

A. Building monitoring in Singapore (Glišić et al., 2005)

The authors report that the objectives of monitoring for this project (forming part of a larger programme of monitoring high-rise buildings in Singapore) were to increase safety, verify performance, control quality, increase knowledge, optimise maintenance costs and evaluate the condition of the structure after an earthquake, impact or terrorist act. The following criteria were set for monitoring and determined the choice of appropriate sensor:

- monitoring was required of critical structural members; i.e. members that in the event of malfunction or failure would generate partial or complete malfunction of the structure;
- sufficiently large numbers of structural elements needed to be monitored to enable local and global knowledge of the structure to be gained;
- monitoring needed to be conducted over the whole life of the structure, including the construction phase; long-term stability and maintaining levels of accuracy and precision were important criteria;
- the monitoring systems had to be designed for structural monitoring (as opposed to detecting local effects); to enable not having to be influenced by local effects and material defects (e.g. air pockets or cracking in concrete);
- sensors were not allowed to be visible and needed to be embedded in the structure;
- The system needed to be performed at ‘reasonable cost’.

The criteria of long system life and stability, not being affected by local effects and embedding determined the choice of the sensing system. The criterion of local effects driving the choice towards a long-gauge sensor. From a fibre optics point of view, this meant employing a SOFO system.

To obtain a compromise between the requirements of monitoring critical elements, taking account of local and global effects and remaining within reasonable budget, monitoring was conducted of a number of ground floor, first and second floor columns (10 columns in total). Figure 98 shows the evolution of strain monitored over a period of 5 years.

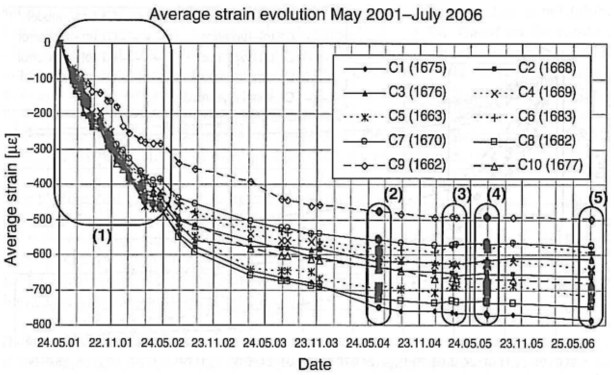


Figure 98. Strain evolution over more than 5 years; (1) construction of 19 storeys; (2) the first 48 h continuous monitoring session in July 2004; (3) before and after tremor monitoring; (4) the second 48 h continuous monitoring session in July 2005; (5) the third 48 h continuous monitoring session in July 2006 (Glišić and Inaudi, 2007).

The monitoring team started pout by analysing individual columns to compare measurements with design. The authors noted particular difficulties with matching the design in some cases and attributed this to uncertainty in what the actual load application was at any given time compared to design assumptions, even though measurements were taken after each storey. This may be attributed to material storage at each storey as the building is constructed. Further difficulties in matching exactly the design numbers were attributed to local effects such as horizontal and vertical column connections, which results in moving the centroid of each column away from the sensor position and complicating back-analysis efforts. This error was quantified as being in the order of 10%. The authors also did not measure temperature, which resulted in a measurement error estimated to be in the order of 33$\mu\varepsilon$for the temperature fluctuation of 10°C. Other contributors to measurement error were strain due to shrinkage and creep.

During the first 12 months of construction, the monitoring followed the design model very closely, but the two diverged thereafter. The monitored strain did nonetheless always exceed the maximum error (defined as 'truth' in the model) and was considered reliable. The drift between measurements and the model later on, while good agreement was achieved during construction indicated 'anomalous' structural behaviour in relation to what was assumed in design. These were attributed following interrogation of the data to: (a) overloading of one of the columns; (b) Creep and shrinkage evolution; (c) stiffness of the second storey three-dimensional structural frame and unknown interaction of monitored and unmonitored columns; (d) unequal foundation settlement in columns and neighbouring cores (this was found to be the most significant effect causing the drift between theoretical model and monitored results); (e) inclination of the second storey. Notwithstanding these effects, the building was shown to remain safe for its residents as it could be shown that critical columns remain far below the required serviceability and ultimate limit cases. Monitoring is nonetheless continuing.

The monitoring campaign also included monitoring the aftermath a tremor in March 2005 by means of a continuous 48 hours monitoring campaign during the tremor.

B. Monitoring of an Arch Dam (Inaudi et al., 1999; Glišić and Inaudi, 2007)

In dam construction, the opportunity exists to use different sensing technologies to enable taking advantage of the strengths of different systems. For monitoring of dams the following parameters are of most significance: (a) strain; (b) deformation; (c) displacement; (e) inclinations; (f) crack detection (integrity); (g) crack opening (for detected open cracks); (h) Temperature; (i) Pore pressure; (j) Seepage detection. Figure 99 shows a typical monitoring strategy for a concrete arch dam to illustrate where different technologies provide the most significant impact from a monitoring point of view.

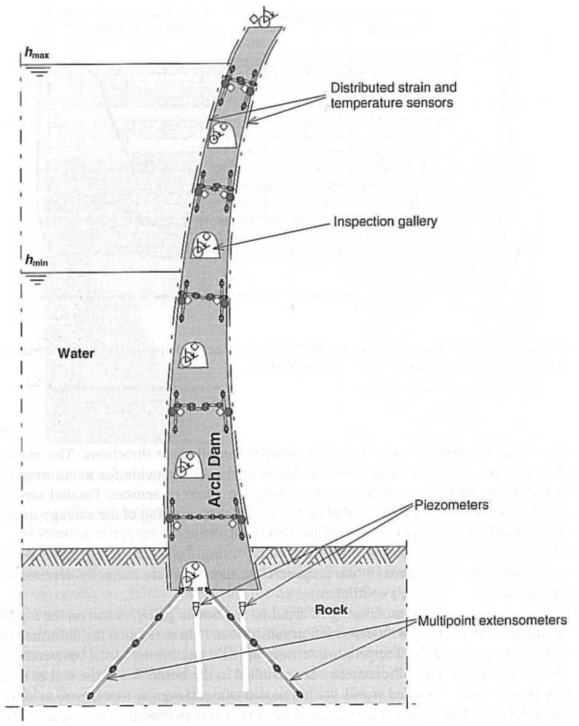


Figure 99. Schematic representation of optical-fibre sensor network in cross-section of an arch dam (Glišić and Inaudi, 2007).

Multipoint extensometers are created by developing a chain of single deformation sensors. Long-gauge deformation sensors can be used to measure in all three directions. If installed through the thickness of the dam these sensors provide for monitoring of the average strain distribution. Parallel installations in horizontal and vertical planes provide for monitoring of the average strain and curvature distributions and evaluation of the deformed shape. Inclinometers provide for the absolute rotation of the galleries and on the top of the crown. Inclinometers combined with deformation sensors should be used to calculate curvature as double integration is known to be less accurate for discrete sensors. Opened cracks may be monitored using SOFO type sensors. Integrity monitoring, however, requires distributed strain and temperature sensing (e.g. BOTDR) and can be installed in areas where the maximum tensile stresses are expected. These sensors will provide information on crack detection, average strain and average curvature monitoring, replacing any discrete deformation and temperature sensors. Distributed temperature sensor provide for seepage detection. Pore pressures are detected using piezometers and accelerometers are used for dams located in seismic areas.

An example where SOFO and distributed sensing was used on the same project is the monitoring of the raising of the Luzzone Dam located on the River Brenno di Luzzonne near Olivone, Switzerland. The dam constructed in 1963 was a 225 m high arch dam with a crest length of 600 m. The dam was raised by 17 m in 1997 to 1998. An important issue of raising the dam was to ensure good bonding between the existing dam and the new concrete poured for the raising of the dam. Incompatibility of deformation caused by early age temperature-generated deformation in the new concrete and later by shrinkage of the new concrete posed a risk of delaminating and loss of structural integrity. Monitoring during and after construction was decided upon, with deformation of the new concrete monitored using discrete optical fibre sensors, while temperature was being monitored using fibre optic with BOTDR technology. The sensors were embedded in the concrete and the results immediately showed the strong effect of drying shrinkage and the dependence on the distance from the concrete surface (sensors close to the air showed much quicker drying and thermal shrinkage than those buried in the centre of the concrete block). The discrete sensor successfully provided an average strain distribution in the concrete block due to drying shrinkage (Figure 100). The successful us of BOTDR temperature sensing is discussed in the section that follows.

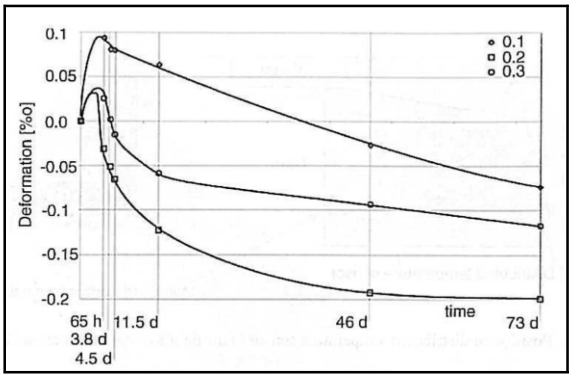


Figure 100. Average strain monitoring results during shrinkage (Glišić and Inaudi, 2007).

Brillouin Optical Time Domain Reflectometry (BOTDR)

Fibre optic sensors with BOTDR are known for their ability to measure temperature and strain over long distances, the ability of distributed sensing the ability to detect local effects (breaks, cracking, local movement, etc). The following case studies show these abilities to great effect.

A. Temperature Detection during Dam Concreting (Thévenaz et al., 1998)

This case study describes the temperature monitoring highlighted in the previous discussion of dam monitoring using discrete and distributed sensors in the same project. In concrete dam construction, it is of critical importance that micro cracking due to temperature gradients be controlled. This is done using the relation between micro cracking and temperatures experienced by the concrete during curing. Thévenaz et al. (1998) report on the use of BOTDR fibre optic sensing to monitor the temperature distribution experienced during the setting of concrete for the Luzzone Dam in the Swiss Alps. The dam was raised by constructing concrete slabs of average size 15 m x 10 m x 3 m thick. An optical telecommunications cable was installed during concreting over the central portion of the largest slab poured (Figure 101). The embedded cable created a dense mat in a horizontal plane, which enabled relaying a two-dimensional temperature distribution of the monitored concrete volume. The monitoring enabled detecting the complete temperature distribution across the slab, indicating a rise in temperature to 50°C in the central area of the slab. The monitoring also showed that it took several months to cool down in this central region, while the outer slab areas rapidly stabilise at the ambient temperature, leaving an outside observer potentially unaware of the heat experienced at the core of the dam.

B. Leakage Detection (Nikles et al., 2004)

Nikles et al. (2004) reported that leakage detection along pipelines is a difficult task and may result in high economic losses, have environmental consequences (e.g. oil spillages) and may result in loss of life due to explosions at leak sites (e.g. gas leaks). Traditionally leakage detection is done by visual inspection to confirm the absence of leaks. For buried pipes, a drop of pressure is defined as indication of leakage, but the reliability of such testing is questionable due to the effects of temperature.

Nikles et al. (2004) reports that leakages from a pipeline introduce local temperature anomalies near the pipeline. Depending on the substance that is being transported the anomalies may be either local warming (e.g. crude oil, brine, heating systems, etc.) or cooling (e.g. gas, water, etc.). The authors describe the use of a BOTDR distributed temperature measurement system used on a 55 km long brine pipeline to automatically monitor leakages to illustrate the effectiveness of distributed fibre optic technology to detect leakage. The detection of leakage based temperature fluctuation is only possible by comparing the measurement results with a baseline measurement to extract environmental temperature fluctuations (which may evolve slowly over time). Nikles et al. (2004) report that techniques available at the time of publishing enabled detection of an initial temperature difference in a localised position, which extends along the length of the sensor as the effect of the leak spreads. Localisation of the leak position is possible to within 1 m of the leak position.

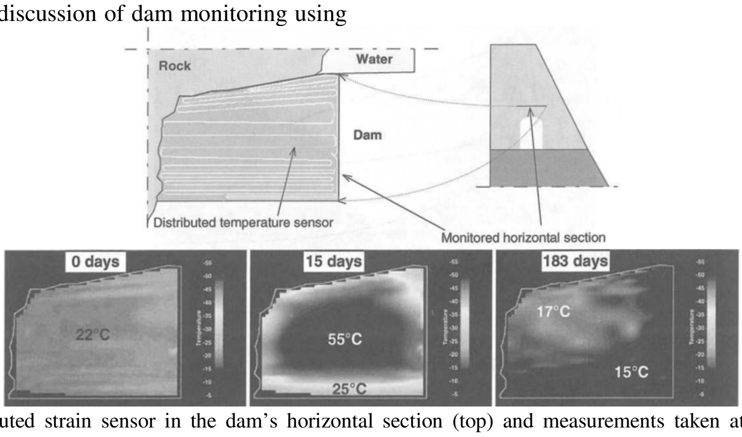


Figure 101. Position of distributed strain sensor in the dam's horizontal section (top) and measurements taken at 0, 15 and 183 days (bottom); (Thévenaz et al. 1998, also in Glišić and Inaudi, 2007).

Construction started in 2002 on an underground natural gas storage facility (at a depth of 1500 m below ground level) in Berlin, Germany. The mining related methods used required hot water for the mining process in rock-salt formation in which the gas storage facility was being constructed. This process produced large quantities of brine (water with high salt content), which was not allowed to be spilled in the surrounding environment. A 55 km long pipeline was constructed for conveying the brine and a leakage detection system was developed to monitor the risk of leakage. The sensing cable comprised of a customised version of a standard armoured telecommunications fibre optics cable for underground applications. The cable included the optical fibres for temperature monitoring, as well as fibres used for data communication between instruments and the control room and additional spare fibres for redundancy.

Nikles et al. (2004) report the placement of the fibres in a trench 10 cm below the brine pipeline at the 6 o'clock position to enable the best opportunity to capture all leakages (Figure 102).

Figure 102. Construction of the brine pipeline, with FO sensor placed at 6 o'clock position (Nikles et al, 2004).

The authors note, however, that the sensing position was a trade-off between the maximum contrast of a leak and the assurance to detect leakages occurring at any point along the circumference of the pipeline. The longest fibre section monitored along the 55 km monitoring configuration was 16.85 km and processing was done every 30 minutes using dedicated software running on a central dedicated PC after receiving temperature profiles from the interrogators. The system was capable of detecting temperature with an accuracy of 1°C in less than 10 minutes across the entire 55 km length. As part of the monitoring system a protocol of alarms, reports and reset and restart measurements were developed and were integrated into the monitoring software.

The brine was injected into the pipeline at a temperature of approximately 35°C, creating a temperature gradient during flow along the pipeline of 8°C. The pipeline itself was buried at a depth of 2 m to 3 m below ground level, which limited seasonal temperature variations to only 5°C. This meant that any substantial temperature increase would therefore be associated with leakage, even at slow leak rates. The pipeline was put into operation in January 2003 and July 2003 produced the first leak related to a local temperature increase of 8°C, which triggered the automatic alarm (Figure 103). The leak was caused accidentally by third party excavation work conducted close to the pipeline.

The success of the leakage detection system enabled Nikles et al. (2004) to foresee possibilities for temperature monitoring in civil engineering, the oil and gas industry, power plats, fire detection and other associated applications. The ability to monitor over several kilometres (also see Nikles et al., 2005 where the ability of sensing is reported to have been extended to enable monitoring across hundreds of kilometres) and being able to detect temperature anomalies at 1 m resolution in reasonable time is very attractive. Nikles et al. (2004) report that leak rates as low as 50 ml/min have been detected.

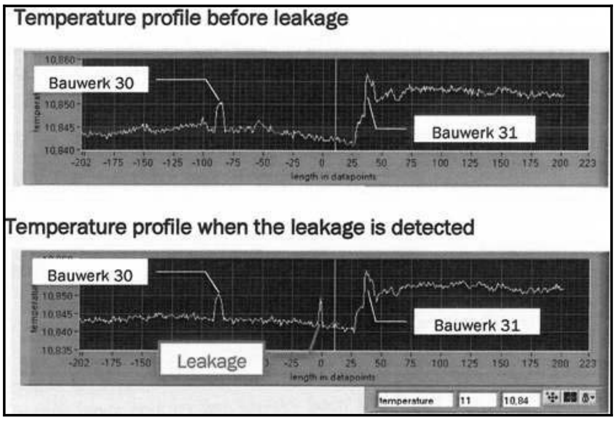


Figure 103. Measured profile before and after leakage. Vertical scale represents Brillouin frequency shift, while the horizontal scale represents the length of the monitored section (Nikles et al, 2004).

C. Extended Distance Pipeline Monitoring illustrated for Blockage Detection (Nikles et al., 2005)

Nikles et al. (2005) reported that the effects of hydrocarbon blockages in gas pipelines by hydrate or wax formation, as well as ice plugging in cold waters increase with pipeline length through the effects of cooling. The problem is significantly greater for flows in deep water and remote sea locations. Currently there are ways of melting hydrate or wax plugs (if detected, but takes a long time), but ice plugs could (at the time of publishing the paper) not yet be detected adequately. This generally results in portions of a pipeline having to be abandoned, or at least large portions to be cut out. Nikles et al. (2005) describes a method for extending the reach of current BOTDR interrogation systems to in excess of 250 km without having to compromise performance. This system could be combined with a 'Smart Pipeline' detection system to enable the pipeline owners to locate and accurately characterise interruptions in hydrocarbon flows, enabling local mitigation measures to be applied to solve the problem. They report that a sensing fibre can be interrogated for temperature and strain with up to 1.5 m resolution using a single instrument over this distance. For long-distance pipelines, this allows active flow assurance measure to be taken by identifying the presence, nature and extent of blockages as they form. This enables pipeline owners to take corrective action on an informed basis, allowing a reduction in the risk ratings of individual pipelines.

Nikles et al. (2005) explains the mechanism for hydrate formation and explains that evidence of hydrate formation at the water/gas interface includes:

- low water content in flow downstream of the hydrate formation location;
- significant temperature rise (4°C to 5°C) due to crystallisation; and
- increasing pressure drop across location as flow is blocked.

The authors indicate that the water content detection can only be done after significant time lag (several hours), while detecting pressure increase due to blockage requires a system of the ability to detect changes to a resolution of the order of 8 $\mu\varepsilon$ (equivalent to a pressure change of 1 bar). In their proposed detection system Nikles et al. (2005) reports that temperature is the main detection parameter, while pressure is used as a confirmation parameter and state that both changes are well within the detection capabilities of BOTDR.

The authors report that by using 'pump-and-probe' techniques extension of previously known systems or remote sensing is possible. The concept is shown in Figure 104(a) whereby a standard fibre optic telecommunication cable is used to bring the pump and probe signals to a low power remote module (denoted DRM) that includes optical signal processing capabilities. For systems where the sensing area is located more than 75 km away from the instrument a dedicated distance extension module (referred to as DRR in Figure 104(b)) was developed. The modules can be cascaded to enable monitoring across literally hundreds of kilometres.

Nikles et al. (2005) report that similar temperature and strain measurement repeatability and accuracy is obtained with this system compared to individual instruments in conventional systems across shorter distance. Measurement time is however slightly longer. The authors demonstrated the high repeatability, high accuracy, capability of detection localised temperature measurement changes in a laboratory setup where the complete fibre length was 125 km, with the DRM located 100 km away from the measurement locations. The authors warn, however, that the selection of fibre cable and the fibre integration is of primary importance to enable a proper measurement analysis. The performances achieved during the experiment are summarised in Table 25.

Table 25. Performances achieved during laboratory experiment.

DRM placed at 100 km (similar performance up to 150 km)	
Length of measurement portion where experiment changes were applied	25 km
Spatial resolution achieved	1.5 m
Temperature accuracy	±0.5°C
Strain accuracy	±10 με
Measurement time	5 min

Although the system was proven in the laboratory with a 125 km fibre length using a single instrument, Nikles et al. (2005) reported their confidence that the same quality of results could be obtained at 250 km by simply cascading DRR modules. This would nonetheless result in increasing acquisition time and additional noise due to amplification required. Using this philosophy, the authors go further by stating that pipeline monitoring of up to 500 km is possible by placing BOTDR interrogators at each end of the pipeline. The authors conclude that the system can be installed to new pipelines with minor incremental cost or time implications.

D. Monitoring reinforcing bars during tunnelling (Thévenaz et al., 2004)

Reinforcing bars are commonly used to reinforce the tunnel face during tunnelling in potentially unstable soils. Thévenaz reported the use of reinforcing toll as sensing elements by instrumenting a reinforcing bar (referred to as a 'Smart Pipe' after instrumentation has been added). The bar was instrumented using BOTDR technology to enable detection of soil movements in real time. By monitoring several pipes, a complete view of the movement behaviour of the tunnel face could be obtained using a single interrogator. The authors report that a fibre optic fibre was placed longitudinally along a pipes placed at critical locations of the face of the Ulsan-Kangdong tunnel in South Korea. The system was used to predict the behaviour of the tunnel during and after excavation by calculating pipe stress and displacement from strain measurement.

It was found that most of tunnel deformation occurred within 1 day of tunnel excavation. This presented the Smart Pipe system with some significant advantages over other systems as it informs the state of the tunnel in practically real time.

E. Long-term Structural Monitoring for Bankside Development, London (UK)

Sensornet and Bennet et al. (2006) reported separately on the installation of distributed fibre optic sensors in piled foundations during construction of the Bankside 123 development, a major building development in London during 2004. The installation was carried out as part of the RUFUS program (Re-Use of Foundations for Urban Sites). The objectives of the installation were to analyse loading effects of piles during construction of the building, to investigate the long-term strain and stress effects in foundation piles and to investigate whether the foundations could be re-used in future. Since foundations are a major cost component for large buildings, it would be beneficial to be able to reuse them especially since buildings are generally designed for only a 30-year design life.

At the time of this project, data was mostly gained from vibrating wire strain gauges. Distributed sensing using BOTDR fibre optic sensors provided the advantage that the sensors are designed with a life of more than 30 years (and so would remain stable over the life of the building). A further advantage was that with distributed sensing data could be produced at 1 m intervals using a standard telecoms fibre as the sensor. The fibre optic sensor used by Sensornet at Bankside also provided the added benefit of measuring both strain and temperature simultaneously, but independently, without cross-sensitivity experienced by other instruments. To achieve the same discretisation with point sensors like vibrating wire strain gauges would be very costly and would require a large number of sensors.

The Sensornet and Bennet et al. (2006) teams attached the fibre optic sensors to the reinforcing cages ensuring that any strain experienced at the outer layers of the sensor cable are transmitted to the optical fibre, whilst also ensuring sufficient robustness of the sensor to withstand construction forces. Sensornet achieved this with their DTSS sensor while Bennet et al. (2006) protected the sensor with an epoxy coating upon affixing the cable to the rebar. Figure 105 shows installation of the BOTDR fibre optic sensor by Bennet et al. (2006). Initial measurements were made in 2004.

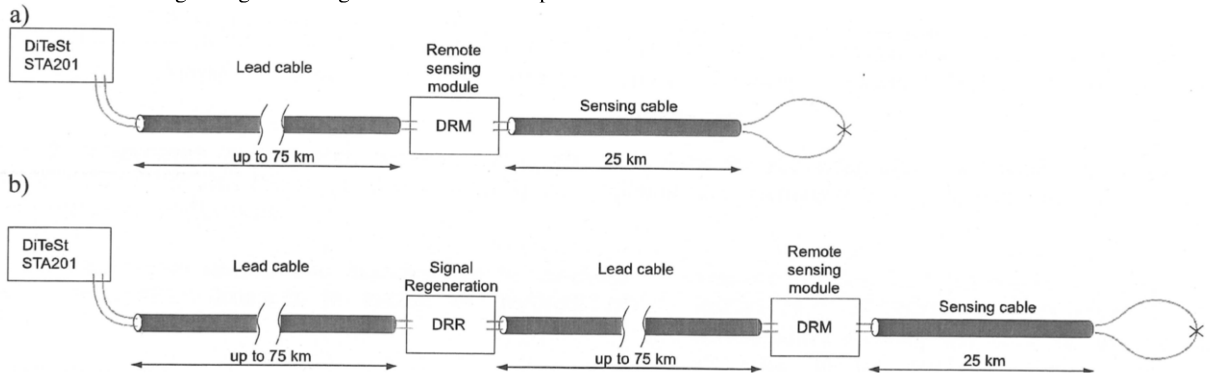


Figure 104. Schematic representation of remote monitoring using repeaters and remote generation for optical signal generation and processing; (a) Up to 75 km; (b) More than 75 km. (Nikles et al, 2005).

Figure 105. Installation of the BOTDR Fibre Optic sensor to the pile reinforcing cage at the Bankside Development.

Apart from the strain measurement, Sensornet also reported the additional ability of the fibre optic sensor to detect the temperature profile during concrete curing. They were able to detect the temperature profile even as the concrete was being poured and could even distinguish between different temperatures of different batches of concrete.

F. Detecting the intricacies of pipeline deformational behaviour (Vorster, 2005)

Pipelines in practice are usually classified as either continuous or jointed (e.g. Attewell et al., 1986). In reality it is usually not straightforward to predict whether a pipeline, comprising various individual pipe sections coupled by joints of specific characteristics, would indeed always react as perfectly jointed where the focus is usually primarily on joint rotation and joint pullout instead of strain (e.g. Attewell et al., 1986; Bracegirdle et al., 1996). The possibility exists that it might react as a continuous pipeline initially, before gradually breaking into smaller sections as joint resistances, axially and in rotation, are exceeded. Capturing a continuous strain profile is invaluable to pinpoint potential localised effects such as joint rotations or joint deformations and non-uniformly distributed soil-structure interaction loads which impact on pipeline behaviour. Practitioners frequently encounter this problem of detecting the intricacies of pipeline and pipe-soil interaction behaviour. Fibre optic sensing with BOTDR provides a means of detecting both local strain changes at joint positions as well as providing the strain profile along individual pipe sections. This capability increases understanding of pipeline behaviour, which is needed for decision-making processes regarding pipeline safety.

To illustrate the use of fibre optic sensing with BOTDR, Vorster (2005) presented a case of a disused portion of a large diameter high pressure water main in London (pipe axis 1.1 m below ground level in made ground and Terrace Gravels) instrumented by means of a dual sensor system to detect the effect of installing a 2.5 m diameter pipe jack at a depth of 11.8 m below ground level in London Clay. The effect of pipe jacking in terms of ground movements is similar to those normally associated with tunnelling (e.g. Mair and Taylor, 1997). The sensor system comprised a fibre optic sensor with BOTDR to measure strain at the pipe crown and sleeved settlement rods to measure pipe crown settlement. Since axial strain caused by pipe-soil interface shear is generally found to be small (Vorster, 2005), the measured pipeline strains could be related mainly to bending strain, εb, and constant axial strain components caused by individual joint behaviour. In conjunction with centrifuge model tests at Cambridge, Vorster (2005) showed that these constant strain components due to joint behaviour could be attributed to:

– loss of rotational resistance;
– axial joint movement; and
– ‘locking’, whereby adjacent pipe sections ‘locked’ onto each other, creating a stick-slip situation leading to increases in compressive or tensile strains being observed. These conditions are normally not distinguished by conventional point sensing systems.

The combination of strain response, pipeline settlement (to estimate increases and reductions in εb due to loss of rotational resistance at joint locations) and existing methods of analyzing pipe-soil interface shear (Attewell et al., 1986), in conjunction with the evaluation of possible local interaction mechanisms (Vorster et al., 2005) provided a complete picture in which local joint effects could be quantified. This provided the opportunity to use the measured BOTDR strain profile for back-calculating pipeline deflection and incorporating local joint effects as boundary conditions for the integration process.

A schematic outline of the instrumented portion of the pipeline, showing the distribution of pipeline joints in relation to the pipe jack centreline and the local geology, is shown in Figure 106(a). The pipeline comprises 942 mm outer diameter, mostly 4.6 m long, concrete lined steel cylinder (LC) pipe sections, connected to an 800 mm outer diameter steel main. The pipe jack centreline was located 3.35m west of the concrete-steel pipe joint. The joints of the LC pipeline portion are similar to standard ‘unrestrained’ bell and spigot joint types (Figure 106(b)). The joint comprises steel bell and spigot rings with a rubber O-ring gasket, which is compressed between the two steel rings during assembly to provide a watertight joint. The annulus between the two pipe sections on the outside was filled with mortar, while on the inside it was left unfilled.

Although ‘unrestrained’, this type of joint is usually regarded as ‘rigid’ by pipe manufacturers, compared to ‘flexible’ joint systems such as rubber gasket joints used in cast iron pipelines. However, since the mortar filling is unreinforced and has a finite tensile resistance, and the annulus on the inside of the pipe remains unfilled, one might expect the joint to have at least some ability to rotate and move axially, which would influence its ability to transfer moment and strain across a particular joint. The transition from ‘continuous’ (strain transferred across joints) to ‘jointed’ (a strain discontinuity formed due to joint movement) depends on the strength characteristics of the joint (especially the mortar filling material which acts as a binder between two adjacent pipe sections) and the joint loading (Vorster, 2005).

A schematic layout and details of installation of the sleeved settlement rods (monitored by means of precise levelling) and the fibre optic sensor is shown in Figure 107. Complete detail of the monitoring system is provided in Vorster (2005).

The field experiment showed that fibre optic sensing with BOTDR technology is suitable for detecting the complete strain profile along a pipeline. Even local effects such as loss of rotational and/or axial resistance at joints can be detected (Figure 108).

This aids in the interpretation of the state of a buried pipeline, taking account of the intricacies of joint behaviour, pipe-soil interaction and interaction between individual pipe sections when the pipeline is subjected to ground movement. The dual strain and settlement monitoring system employed in the field trial allowed the BOTDR strain measurements to be verified against settlement measurements made by means of settlement rods and precise levelling. By applying known effects of joint behaviour on the measured strains, the BOTDR strain profile could be double integrated to infer settlement. The inferred and measured settlement profiles corresponded very well, confirming the virtue of the fibre optic strain sensor with BOTDR to provide both strain and inferred settlement data. It also provided confidence in both the settlement and strain monitoring systems used.

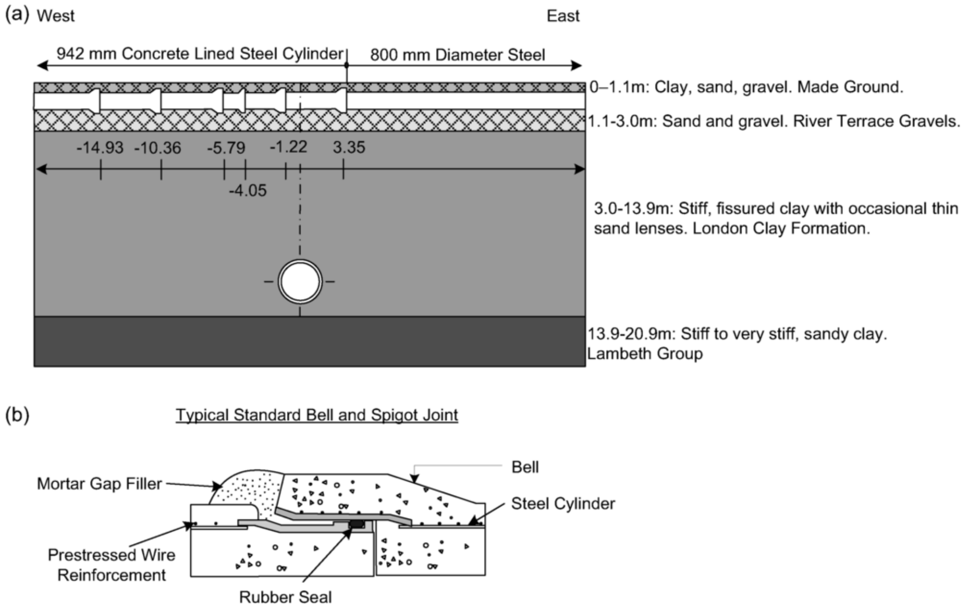


Figure 106. (a) Geology and joint locations relative to pipe jack centreline. (b) Typical LC pipe joint detail (Vorster, 2005).

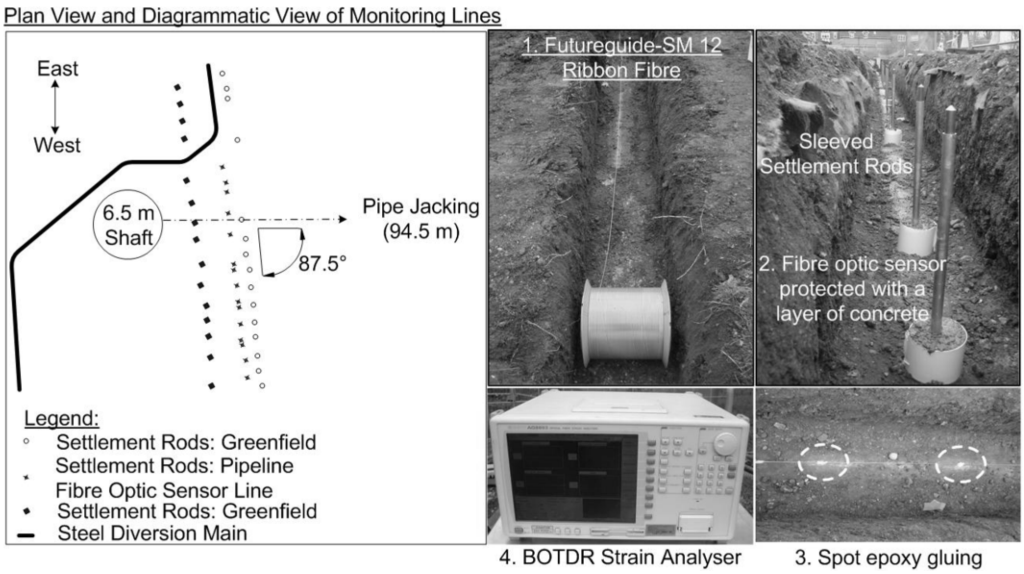


Figure 107. Instrumentation layout and photos of the sensor system during installation (Vorster, 2005).

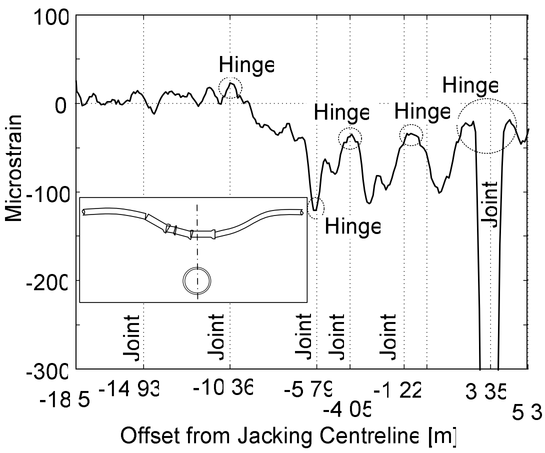


Figure 108. Typical result indicating joint formation in a prestressed concrete pipeline (Vorster, 2005).

G. Monitoring State Changes in an Underground Mine (Naruse et al., 2007)

Naruse et al. (2007) explains that blasting and excavation in underground mining lead to changes in the mine 'state' (state refers to prevailing temperature, deformation and stress). These state changes frequently lead to serious accidents and sometimes death to workers. Previously existing systems of detecting changes in the state of an underground mine include ground surface subsidence monitoring using radar interferometry, using satellite image data and monitoring changes as observed from the analyses of images taken from the mine. Naruse et al. (2007) proposed to improve on current monitoring systems by being able to detect deformation or damage in the mine itself and detecting movement of the rock mass directly. Detection of rock mass deformation was aimed to be achieved by monitoring the underground infrastructure of a mine using fibre optic technology. The authors report that in the past attempts to monitor underground mining infrastructure using fibre optic technologies included:

- monitoring temperature in underground coal mine roadways using distributed sensing (Willet et al., 1995);
- monitoring rock deformation using a distributed fibre optic sensor buried in rock (Chai et al., 2004);
- monitoring pillar stress in a coalmine using OTDR technology (Heasley et al., 1997). The detection system was based on interpreting bending of the optical fibre and optical loss produced by such bending. It was found however that this method of detection was restrictive since the number of units monitoring bending and optical loss is determined by the total optical loss (which increases as the number of units increases) and;
- monitoring displacement, strain and temperature in an underground mine using fibre Bragg-grating (Fisher et al., 2001). The authors explain that in this application, the installation number was restricted because power spectra reflected from different FBGs cannot be distinguished due to overlapping when the number of FBGs increases.

Naruse et al. (2007) set out to attempt to solve the problem of sensing changes anywhere across a large extent of a mine by using BOTDR technology due to its ability for long-distance and spatially continuous measurement. They devised a monitoring system at an operating underground mine to monitor changes in deformation trends produced in a ventilation tunnel of the El Teniente mine (specifically in the Diablo Regimiento area) over a period of half-a-year. The El Teniente mine is the world's largest underground mine (producing 130 000 tons of copper ore per day) located at the foothills of the Andes mountains. Naruse et al. (2007) explains the mining method of pre-undercutting panel caving and how the process of mining produces an imbalance of stress surrounding the undercut face in front of the undercut zone (Figure 109). The mechanism they identified as creating the imbalance is explained as follows:

- when rock pillars are undercut, the load in the pillars reduce, with a subsequent reduction in stress in the rock below the pillars;
- on the other hand, stress in rock pillars that have not been undercut, increase due to redistribution of stress;
- this results in higher stress in the undercut zone and variable stress conditions closer to the undercut face, which is advanced in 45 m to 50 m stages;
- in addition, large-scale ore extraction above the extraction area changes as production continues;
- these processes result in complex and changing stress conditions across the mine and the deeper lying infrastructure.

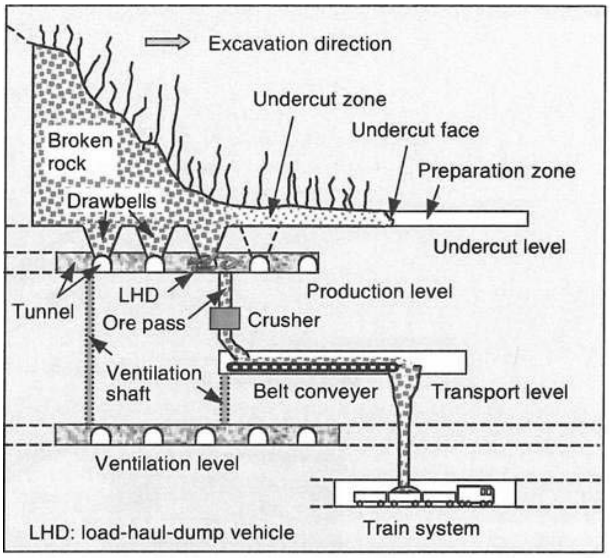


Figure 109. Vertical cross-section of the Diablo Regimiento area (Naruse et al., 2007).

Naruse et al. (2007) installed monitoring system (Figure 110) to detect the changing state due to these changes in 201 m long section of a 5.2 m wide x 4.6 m high underground mine ventilation shaft with arch-shaped roof. The detection position was chosen to be roughly perpendicular to the undercut face. The authors also highlight some difficulties in the choice of monitoring position due to mine operations and infrastructure at other locations that could not be interrupted. They also describe the method of installation and temperature compensation. The trial was conducted between May and November 2005 during a period, which included the process of undercutting and draw bell construction, excavation area expansion and large-scale ore extraction. From the results of the trial, they report that the strain-based BOTDR monitoring arrangement used could detect the deformation of the ventilation tunnel caused by stress changes surrounding the undercut face and caused by large-scale ore extraction. They observed deformations corresponded with the progress of these mining activities and concluded that the system was a practical solution and that BOTDR distributed fibre optic strain sensing is promising for underground mine monitoring.

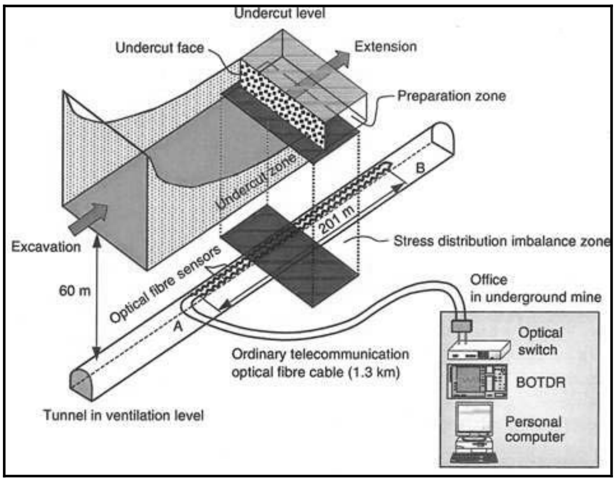


Figure 110. Overview of the underground mine monitoring system at the El Teniente mine (Naruse et al., 2007).

H. Dynamic Integrity Monitoring for Rail Infrastructure (Sensornet, 2005)

Distributed fibre optic sensing technologies such as Raman OTDR and BOTDR are known to have the limitation that it takes longer than technologies such as FBG to interrogate measurements (e.g. Nikles et. 2005). As such, it is generally not suited for use in dynamic detection systems. Sensornet reported recently, however, that, in association with SAOM Consultants and the Korean Railway Authority (KORAIL), the first ever-distributed dynamic strain measurement was achieved in 2005 on the Korean Train eXpress (KTX) railway track in Daejeon, South Korea. The objective was to monitor 60 m of recently repaired track using distributed fibre optic temperature and strain sensor (DTSS) while a KTX train passed over it. The KTX is a high speed train, capable of travelling at 300 km/h, with a total length of 338 m, weighing 771.2 tons fully laden (80 kN wheel load on wheels spaced 500 mm apart). The DTSS is able to measure fully distributed strain at rates of up to 10 Hz, allowing detection of rapid deformations or movements in structures. This must be viewed against traditional Brillouin-based systems, which are usually only able to complete a measurement in 20 seconds to 20 minutes.

The sensor was bonded to the lowest part of the rail web (rail was 140 mm deep, 140 mm wide flange and 10 mm thick web and flange) to ensure ease o f installation and high sensitivity to flexural movements (Figure 111).

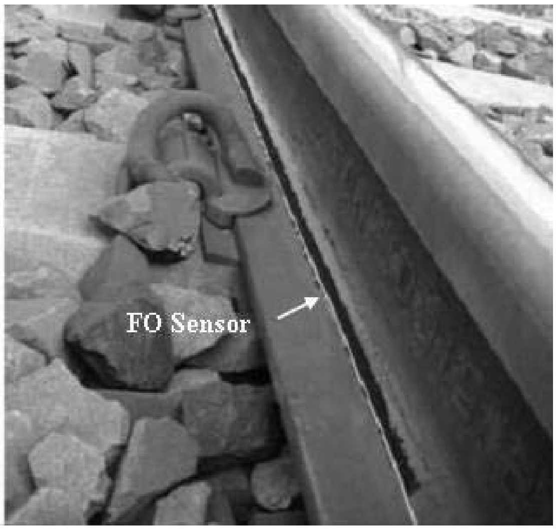


Figure 111. Fibre-optic sensor attached to the rail (Sensornet, 2005).

To mitigate potential problems with expansion occurring during winter and summer an expansion joint was inserted in the track. The team also replaced one of the concrete sleepers with a wooden sleeper to ensure greater flexibility at the expansion joint. The system had a communication lead of 190 m and the monitored portion of the track was located 260 m along the length of the sensing cable, resulting in a total distance of 450 m between the location of sensing and the interrogator. The track was monitored as the train decelerated into the station, taking approximately 1 minute to pass over the monitored track (approximately 20.3 km/hr).

The system was able to distinguish on a coloured strain map the difference in strain on the existing portion of track (30 to 50 με) compared to the repaired portion of track (90 με); initial strain before the train arrived measured between 0 and 10 με. A theoretical assessment of expected strain yielded a strain prediction of 100 με, which was in good agreement with the 90με measured.

Based on the monitoring results the project team were able to conclude that, even though the newly repaired portion deflected more than the surrounding track, the rail flexure was well within operating guidelines. The DTSS had a spatial range of 10 km at the time of conducting the trial, establishing the opportunity for monitoring longer sections of track, targeting higher risk sections such as bridges, areas of repaired track and areas at risk of subsidence.

New fibre optic devices being developed with possible use in geotechnical Engineering

Romashko et al. (2007) reported their development of a highly sensitive and fast-adaptive interferometer based on a dynamic hologram in semi conductive crystal and multimode fibre as a sensor. The sensor allows for detection of ultra-small displacements (0.1 nm, or 10-4 με), while the adaptive properties of the dynamic hologram eliminate all unwanted low frequency influences. Romashko et al. (2007) are hopeful that this new development would hold much promise for long-term monitoring of ultra-small vibrations and dynamic deformations in the industrial environment.

7. SUMMARY AND CONCLUSIONS.

An attempt was made to review three core activities of geotechnical engineering: *the prediction, the monitoring and the evaluation of performance* of some geotechnical structures. These activities are jointly developed in what is currently referred to as *Interactive Design*. It has been noted that the interpretation of the observed performance and the detection of subtle deviations (which some times are not so subtle) is largely taken for granted and overlooked. This is supported by many cases of inability to avoid the collapse of some geotechnical structures.

Within this report, four types of geotechnical structures were contemplated, each with its particular type of response, conditioned by the stress paths and singular failure modes involved. These were *Foundations* (mainly deep), *Earth Fills, Supported Excavations and Tunnels.* Accordingly, each section of this report was named after each type of structure. Moreover, each section covered a description of the typical response of each geotechnical structure, an evaluation of response anticipation, a review of procedures for measuring and for performance evaluation and a discussion on the specificities of application of Interactive Design, on each type of structure reviewed. A final section is provided on geotechnical instrumentation, in which the basic requirements for instrumentation planning and selection of instruments were reviewed. Also, new trends and recent monitoring developments are discussed.

In the section on *Foundations*, which addressed mainly deep piles, it has been noted that settlement based design is not routinely adopted as a design method. In turn, ample factors of safety are applied to considerably scattering bearing capacity factors, provided by available ultimate states theories, in order to control settlements, by limiting the stresses in the underlying soils. Moreover, it was noted that the practice is dominated by a widespread conservatism, in which codes and standards act as an inhibition rather than an incentive to innovation. Not surprisingly, *Interactive Design* is seldom used, meaning that a change in a foundation design, when a structure is partly built, is normally a difficult task. As a result, project specifications often do not call for monitoring of the performance of the foundation.

Proper *evaluation of prediction* of deep foundation performance usually requires model tests or full scale field testing. This is particularly the case, when the design is pushing the boundaries of accepted practice and there is a need to satisfy regulatory requirements. For that purpose, testing using Osterberg cells has been found to provide valuable data for highly loaded piles. Regarding *evaluation of performance* of deep foundations, differential settlements are normally the key parameters, as total settlements are not usually an issue, except for assessment of bridges ride quality and service connections to buildings. The differential settlements expressed in terms of distortions, deflections and tilts and limiting values to support performance evaluations are provided according to the type of structure at hand and the type of potential damage involved. Projects that do come to the fore with settlement measurements are often those with problems and, regrettably, in many cases the information is not published due to issues associated with litigation. Finally, two selected case histories were presented (towers in Dubai and Melbourne), where the foundation designs of the buildings were mainly based on settlement.

The section on *Earth Fills* refers mainly to the fills built on soft cohesive soils, in which the applied earth fill load induces stress increments exceeding the soil pre-consolidation stress. The earth fill response will, thus, depend largely on key parameters of the soft ground that control its compressibility and strength. The definition of these parameters is highly dependent on site investigations. On balance it is concluded the quality of soft ground site investigation has regressed in recent time, despite novel laboratory and field testing techniques to improve it, such as radiography screening of tube samples, constant rate of strain testing for soil compressibility assessment, piezocone profiling, in situ undrained strength assessment by competing techniques as vane and piezocone testing. Reference was made to rates of field consolidation, in sedimentary soft soils, much higher than that anticipated by coefficients of consolidation provided by laboratory and piezocone testing. This is frequently related to the occurrence of unnoticed thin sand layers within the soft soil, reducing drainage paths.

For a true *evaluation of prediction* of the performance of an earth fill to be built over soft soil, an instrumented trial fill is likely to be needed. Instrumented trial embankments are particularly important when there is high variability in soil conditions or when there is no previous experience. Models used for the prediction can be revised and recalibrated with back analysis of the trial data. Accordingly, *Interactive Design* can be fully applied to earth fill constructions with observations being made at the early stages to decide future construction activity. The performance can be continuously checked and the design tools and models can be continuously updated as required whilst the future performance is re-assessed. When strength increase is an issue, the use of CPT tip resistance may supplement traditional field monitoring data. Regarding *evaluation of performance* of earth fills on soft ground, if the model in use does not satisfactorily fit all the observed and considered reliable data, both the model and the relevance of the data to the particular model need careful consideration. Any attempt to massage the soil model to suit the data obtained or to dismiss data which does not fit the chosen model, implies a lack of confidence in either the data or the design model. Such lack of objectiveness should be assiduously avoided. Differential displacements as well as absolute displacements as much as distortions are usual *performance indicators* for earth fills. Limiting displacement values were provided according to the type of structure to be placed on the earth fill and to the criteria applicable. Case histories reviewed included a river diversion embankment built over loosely dumped fill, from overburden stripped for coal in Victoria, Australia, and an embankment dam built near Istanbul, Turkey, over thick soft soils. In the first case the project required the construction of four culverts for conveyors, with fills up to 45 m above the base of the culverts and with varying foundation conditions. The observed and predicted culvert settlements and the possible reasons for the variations are discussed. In the second, settlements and pore pressure observations with time, indicated rates of consolidation much faster than that anticipated with coefficient of consolidation obtained in the laboratory.

In the section on *Supported Excavations,* factors controlling their behaviour were reviewed. Stability in general is governed by classical bottom heave mechanisms in soft to medium stiff clays and hydraulic uplift stability in frictional soils. In stiff clays, or sands above the water table, stability is normally not an issue, as long as struts or anchors are properly designed to take the anticipated loads. Strut loads and wall displacements depend on the stability condition. In soft clay excavations, the normalized sum of maximum strut loads decrease rapidly with increasing bottom heave safety factor. On the other hand, the maximum wall deflection increases rapidly when the apparent bottom heave safety factor decreases. Moreover, wall system stiffness does not seem to significantly control displacements as previously thought. The width of the excavation as much as the stiffness of the soil may have an important role on displacements magnitude. Apart from the stability condition, other factors that may impact on the induced displacements are many and were discussed. Generally speaking the maximum ground settlement varies from 0.5 to 2 times the maximum lateral wall displacement. The lateral extent of the ground settlements is more dependable on the depth of the firm stratum than on the excavation depth. Notwithstanding this, other factors affecting the relation between settlements and lateral wall movements were listed and discussed.

For *measuring excavation performance*, a strategic instrumentation planning criterion was presented, defined as a function of monitoring purposes (classified into 7 categories), by selecting the type of measurement to be taken, on the basis of a weighted scale of utility, or usefulness, ranging from 1 (least utility) to 5 (highest utility), attributed to each purpose category. A similar approach could possibly be extended to other geotechnical structures with minor adjustments. A brief review was presented on instrumentation novelties applied to supported excavations, including fixed inclinometer strings on walls, the potentials of fibre optic sensing for movements monitoring, automated total stations for settlements, the measurement of building or surface settlements using differential SAR interferometry (DInSAR), using orbited satellite radars imagery, with accuracy of 2 to 3 mm or, on a more local level, using laser light array scanning (LiDAR), with accuracy of 1 to 3 mm. Finally, the possibility of real-time fully automated monitoring of essentially all parameters of interest was also discussed, using communication and data base systems presently available.

Regarding *performance evaluation* of *supported excavations* or *tunnels,* a particular parameter to be considered is the *acceptable deformation* on neighbouring surface buildings. Accordingly, an example of settlement criteria used for excavations was presented, in which building settlements, rotations, hogging and sagging ratios were used as *performance indicators* and limiting values were defined for a particular excavation project (Taipei Metro), for distinct building types. Moreover, damage categories defined by a five levels severity scale were reviewed, relating them to a limiting tensile strain (horizontal) and to the building deflection ratio, and relating them to lateral strain and angular distortion. Modern FEM analyses are commonly used for *performance prediction* of associated damages in buildings adjacent to excavations. However, it seems that numerical capabilities are far more advanced and incorporated into design practice than the determination of relevant soil parameters. This is particularly true in relation to the higher soil stiffness in unloading/reloading than for first time loading, which leads to over-conservative damage estimates when one fails to account for its action.

For *supported excavations* as much as for any other geotechnical structure, measurements are by themselves of little value, unless realistic acceptance criteria in terms of alert and alarm levels have been set prior to construction. Implementing remedial actions may be easier said than done, considering that the time element is important, and that safety of workers must be ensured. A decision to immediately implement such remedial actions, on the spur of the moment, is a major challenge to the parties involved and requires a number of preconditions. Displacements are generally preferred for monitoring but are more difficult to relate to the potential collapse of struts and anchors. If displacements or loads at any stage of the excavation differ significantly from what is predicted, supplementary analyses should be carried out to find an explanation for the difference. It is therefore of great importance that the designers of the excavation are involved and available during the construction phase.

Interactive design is not a well suited approach for deep excavation projects unless it is a long excavation in rather uniform conditions. Elements best suited for interactive design are vertical and horizontal spacing of soil anchors for anchored walls among few others. In long cut-and-cover projects one may start off with a conservative design, and then move to a less conservative direction, provided that the excavation support performs better than originally expected. Finally, it was reviewed the case of the Nicoll Highway supported excavation collapse, in Singapore, in spite of a rather extensive monitoring program and of the successive revisions of the limiting lateral wall displacement. It was noted in this case a not uncommon reaction, also seen in other cases of failure, in not willing to accept the burden of admitting that something is fundamentally wrong. Whenever such a syndrome is detected (what may not be indeed so simple), the minimum to do is to stop the works and make in-depth checks of the design, construction and performance records, preferably by independent and acknowledged experts. The importance to verify and calibrate numerical models coupled with input data from high quality soil data, against well documented case history is undisputable. Proper record of performance is, however, not limited to measurements of lateral wall movements and ground surface settlements. It is surprising to see how few well documented and fully covered performance of supported excavation have been published.

In the section on *Tunnels,* the *typical soil response* was reviewed in terms of stress path and strain developments, for an element around the tunnel under 2D idealized conditions (drainage allowed). It was noted that under good tunnelling practice, a non linear soil response is always expected and soil dilation may prevail for appreciable amount of in situ stress reduction. Soil rigidity and degree of non linearity were discussed as well as soil stiffness degradation, being noted that strain ranges in usual tunnelling conditions, involve non linear elastic or pre-yield plastic strain components, to which conventional soil testing instrumentation in the laboratory (proper for measuring large strains) may not be adequate. The typical soil response to tunnelling was then reviewed, for undrained conditions, being noticed that under appreciable decreases of tunnel internal pressure, negative pore pressure changes are noted both for lightly or heavily consolidated clays. It was further pointed out that, even for an incompressible soil mass, the transient hydraulic boundary conditions of an advancing tunnel heading may explain Laplacean pore pressure changes controlled by the rate of tunnel advance, in an otherwise time independent condition. A review was presented on simple criteria for anticipation of drained or undrained conditions that may prevail under certain extreme tunnelling scenarios of an idealized deep impervious tunnel and of a shallow pervious tunnel. Local and global collapses were discussed and the three-dimensional nature of collapse mechanism reviewed. Finally, the plane strain stability with time of a tunnel, in which the lining action is mimicked by an internal pressure, was analysed. Both for over and normally consolidated clays, the long term stability may or may not be critical, depending on the amount of stress release allowed and on the degree of over-consolidation. It is known from observations and theory that if ground control is good, the undrained changes in pore pressures around the tunnel are likely to be small, compared with other geotechnical structures. In contrast to an open cut excavation, the mean principal stress in the tunnel cover does not decrease as much. Pore pressures are mainly control by shear stress changes. If these are limited, as in good tunnelling practices, the changes in the factor of safety are small after undrained construction, provided tunnel contour is impermeable.

The first standard question which is normally made prior to *Measuring the Performance of Tunnels in Soil*, namely – "Why measuring the performance?" may have distinct answers, depending on the tunnel construction technology involved. For a traditional mining construction (NATM, for instance), one reason to measure performance is the assessment of the stability condition of the ground mass at the unsupported heading (assuming that once it is lined, the tunnel is essentially stable). On the other hand, for a TBM driven tunnel, measuring the performance allows the assessment of the efficiency of tunnel face stability control and of the grouting behind the lining for loss of ground control. The interactive design shows higher potential in a traditional mining construction, while it is limited to optimizing TBM operation and/or slurry or from parameters in the second case. The second standard question – "What is to be measured?" does not depend much on the tunnel construction technology: displacement in the ground are related to the safety of the excavation; displacements, distortions and loads in the lining or in neighbouring structures are related to assessing potential damage and to interactive design. The answer to the third standard question – "How to measure displacements and loads for tunnels in soil?" – is trivial, as field instruments used for other geotechnical structures may be equally used in tunnels. With the exception of fibre optic sensors (FO sensors) and data acquisition systems allowing their automation, it appears that not much novelty in instrument hardware has been introduced recently into tunnel monitoring practice. Accordingly the conceptual design of a FO inclinometers is presented for installation around a tunnel in soil, both in vertical and horizontal directions. The proposed design, that includes Brillouin scattering sensors with an unstrained reference fibre, is yet to be proved in practice.

The *evaluation of prediction* of soil tunnel behaviour is frequently done by comparisons between calculated and measured performance. Prediction is currently performed by numerical modelling. A word of caution was given regarding using these comparisons to detect non conformities in the design or in the construction. Despite this, the need of numerical modelling for proper tunnel monitoring was duely stressed. Difficulties, with modelling for monitoring were discussed, on the basis of some bench marking numerical evaluation for tunnels in soil reviewed in some recent state-of-the-art reports by other authors. The authors of this report share the view of others that numerical modelling of tunnels does require guiding and training, particularly when defining limiting values for field monitoring. The need to "design the tunnel modelling" including mesh design and constitutive representation of the soil was further stressed. A brief discussion was presented on the uncertainty involved in tunnelling performance and on the proper way to handle it in the modelling for monitoring. Uncertainty is partly due to variability of the soil. Despite the advances on probability approaches applied to tunnels, industry still favours deterministic analysis, using averaged soil properties and accounting for variations of parameters by using appropriate factors of safety. Peck in 1995 explained that engineers are comfortable with the current state of practice. Part of the "comfort" referred to by Peck can be attributed to the higher cost in investigations to cover stochastic description of the ground. However, simpler and less expensive tunnel design approaches accounting for uncertainties can be envisaged and reliability approaches used in newer geotechnical areas such as environmental and off shore may be spillback into more traditional areas such as tunnelling.

An extensive review of comparisons between *numerical predictions and actual tunnel performances* was made, complementing an earlier review published almost two decades ago now, totalling more than one hundred cases. Some of the results found earlier were confirmed but some changes were observed during the last two decades regarding results of the earlier survey. Most cases refer to 2D analyses but 3D modelling is becoming popular, yet mainly in the academy, as this type of modelling is still "engineering time consuming". Lambe's (1973) classification of prediction was appended by identifying cases of actual prediction, back-analysis and predictions done using a previously calibrated model. Moreover, a four level comparison rank was adopted, the lowest level 1 applied to comparisons involving just one performance aspect (surface settlements, for instance) and the highest level 4 applied to cases in which all basic response parameters were compared (the complete soil displacement field plus lining loads). As before, no clear correlation was found between the efficiency or certainty of the modelling tools used and soil type, construction method, prediction type, type of numerical simulation and of constitutive model used. Therefore the review was again, reduced to a broad appraisal. The majority of cases (76%) refer to predictions made after the event, with results from field instrumentation already known, most of these cases being better described as a back-analysis. This has the obvious result of biasing the appraisal. The 3D effect of the tunnel advanced in a 2D numerical representation is accounted for by ground stress reduction in 70% of the 2D analyses reviewed. In more than half of the cases reviewed, linear elastic-plastic models in which yield and failure coincide were used. Also in more than half of the cases only one tunnelling performance aspect is investigated (generally surface settlements). Level 4 comparisons were performed in only 12% of the cases. It was noted that in the last two decades the amount of level 4 comparisons decreased considerably. The magnitude of the maximum observed surface settlement is closely matched by numerical prediction in just more than 60% of cases, a poor result if one consider the possible bias present in the data collected, considering that in the majority of the cases the maximum settlement was known prior to the analysis. In more than half of cases the numerical simulations furnished surface distortions smaller than those observed, a feature possibly related to shear strain concentration in narrow zones and the inability of most numerical codes to portray properly shear band formation. The maximum magnitude of the measured horizontal ground displacement transverse to the tunnel is either matched or over-predicted by the numerical models. This may be associated to the assumption of constant amount of stress release (a given fraction of the in situ stresses) applied to all points of the 2D tunnel contour. It appears that results from 3D analysis tend to show better agreement with observed lateral ground movements. New stress release criterion for 2D analysis could perhaps be defined from comparisons of results of 3D and 2D analysis. Different amounts of ground stress release around the tunnel contour may also have a positive effect on the distribution of lining loads, otherwise assessed simply as regular in most of the cases reviewed. On the other hand, the comparison of calculated and observed maximum magnitude of lining loads revealed that lining loads tend to be over predicted or matched by the numerical models and only rarely are under estimated; likewise lateral ground movements. It is suggested that in prefabricated linings a full or partial slip may reduce the estimated maximum magnitude of lining loads. It is also suggested that in sprayed concrete linings, the use of reduced Young's modulus to account for hardening and the occurrence of creep in the concrete when loaded at early age lead to higher loads estimate and lower lining loads measured respectively, possibly explaining the over prediction of lining loads. The best ranked results reviewed in the last two decades refer to 3D FE analyses on a slurry shield tunnel and on a NATM tunnel.

Regarding the *evaluation of tunnel performance,* the limitation of straight comparison of field measurements with predicted quantities was discussed. The case of Pinheiros Metro Station in Sao Paulo, Brazil, was reviewed using the classical ratio measured to calculated elastic displacement at the opening contour for the opening stability evaluations. A conformity condition is accepted when this ratio is smaller than 2 and a near ultimate state condition is defined when this ratio ranges from 5 to 10. A type A prediction made well before the station construction indicated that the rock cavity would have to be stable for crown settlements smaller than 18mm and instabilities were to be expected for settlements between 45 to 90 mm. The station collapsed just after crown movements of only 34 mm were measured. Bearing is mind these difficulties an extensive review of 15 *performance indicators* for tunnels in soil was made, some of then related to serviceability, some to ultimate state, some already in use and others not so much. They are preferably defined as dimensionless quantities for generalizations. Their potential applications and limitations for performance evaluation were discussed Table 19 summarizes these performance indicators, providing their symbols, their definition or concept and furnishing serviceability and ultimate state limiting values wherever possible. Redundancy of evaluation is required due to usually complex ground conditions as well as complex boundary conditions. Users of these indicators could scale the dimensionless parameters applying weights to each indicator and summing up an overall performance mark. The amount of time to assess the indicators does not represent an over increased data analysis and adds substantial technical and management value to the routine field monitoring. The crown settlement to tunnel diameter ratio has limiting values derived from observations in laboratory tunnel model tests and present too wide a range of ultimate state values for assertive practical use. The surface to crown settlement ratio at tunnel axis has an ultimate state limiting value of 1, but caution should be taken when applying it to consolidating or contracting soils upon tunnelling in which the indicator may exceed unity without involving collapse of the tunnel heading. Subsurface distortions have ultimate state limiting values derived also from laboratory model tests and can be derived from plots of crown settlements with distance to the tunnel face. Surface distortions may have limiting values for serviceability and for ultimate state not of the tunnel proper but to a nearby existing structure as presented in section 4 of this report. Changes on the distribution and on the sign of the longitudinal distortion index (LDI) can be related to impending tunnel heading collapse. Associated to changes on LDI distribution, there are changes in the ground displacement vector magnitude and orientation at points close and around the tunnel heading face. Considerable increase in the horizontal component of ground vectors close by the tunnel face is noted when failure is approached. Heterogeneous ground conditions may make this interpretation more complex. The volume of surface settlement as a percentage of tunnel volume (%Vs) has been related empirically to the quality of construction. However, whenever %Vs is related to transverse distortion at ground surface, on the grounds that a good quality tunnel construction would likely represent smaller risks of damages induced on existing structures, inconsistencies appear. Expected ranges of loss of ground for distinct tunnelling technology and ground type gathered by practice, can define limiting values for serviceability and for ultimate state. This indicator operate well for assessing ground control condition near the excavation but says little about it impact on the surface, which depends on volume changes in the tunnel cover. Contracting porous soils or consolidating soft soil may enhance surface settlement and associated damages. Limiting lining distortions ranges were compiled from field observations and can be taken as serviceability limiting values for different soils type. If the lining ground contact is poor, with concentrated ground loads or with voids between soil and lining, the limiting distortions may not be valid. Reference lining loads taken as a fraction of the overburden range usually between 25% to 75%, a range that can be taken as limiting serviceability values. Whenever measurements lie outside this range, a non conforming condition may be present and requires investigation to explain the deviation. The maximum lining load, defined from a uniform pressure causing collapse by buckling can be estimated from the theory of elasticity. This value can provide unsafe estimates of lining loads if the ground lining contact is poor. A dimensionless crown displacement (U_c) was defined. Serviceability and ultimate state limiting value for U_c were proposed on the basis of drained and untrained tunnel model test results in reconstituted soils. The ultimate state criterion proposed was validated by field measurements taken in some tunnels just prior to collapse. Maximum tunnel face extrusion can be estimated from a 3D numerically derived elastic solution obtained from parametric 3D linear elastic analyses, for full face shallow tunnels, for certain range of parameters and conditions. The solution has been tested in some tunnels and it is estimated that a non-conforming condition is present when the measured extrusion is larger than one to two times the calculated extrusion. The same parametric 3D FE linear elastic analyses described have been used to furnish an estimate of the tunnel crown settlement at the tunnel face as a function of tunnel diameter, depth K_0 and the in situ tangent modulus of the soil at crown elevation. The solution was tested in more than 50 documented tunnel projects, being found that a non-conforming (serviceability limit?) condition is attained for measured crown settlements greater than the calculated value. More importantly, an ultimate state limit is likely to be developed when the measured crown settlement at the tunnel face is greater than two times the calculated settlement. Similarly, the springline transverse displacement at tunnel face can also be estimated using the same 3D finite element derived results. This solution has been tested in more than 20 tunnel projects and it was found that a non-conforming condition (serviceability limit?) may be present whenever the measured lateral movement of the ground exceeds the calculated vale.

Furthermore, a safe ultimate state may be reaching when the measurements are twice as large as the calculated value. The 3D FE parametric elastic analyses yielded also a solution for estimate of the maximum longitudinal distortion, for a point located at a distance 0.3D above the tunnel crown. A limiting serviceability condition is said to occur when the measured distortion from a deep settlement point is greater than the calculated value. Finally a special performance indicator was proposed: it is related to poor ground control condition associated to excessive overcutting or inefficient grouting behind prefabricated linings. It was derived from 3D FE parametric elastic analyses combined with observations gathered from more than forty tunnel projects. Limiting dimensionless crown settlement increment (ΔU_c), after installation of prefabricated lining was defined for serviceability (0.3) and for ultimate state (1.0). The latter could have anticipated ground collapses behind precast-liners in some tunnel projects quoted. As it can be noticed, most of the tunnel *performance indicators* reviewed were derived semi-empirically. Many of them have been in use by some, over the last 20 years, with confirmed good results. Their nature, however, require constant hindsight and adjustments to better define their limitations and ranges of validity.

Interactive Design is an ideal process design approach for traditional tunnel construction, with sprayed concrete lining. Even in a TBM contract with prefabricated lining, interactive design can be applied for optimization of machine operation (applied torque, advance jack thrust, cutting rotation speed), face control (face pressure, slurry or foam mixture, face window openings, direction of cutting face rotation), grout control (overcutting, grout mix, grout pressure).

However, in NATM tunnels the potentials for this design approach are higher, particularly in ab initio applications, through which considerable savings may be reached in simplifying excavation sequences, in reducing ground conditioning at the tunnel face, in increasing the depth of excavation advance, in increasing the distance of lining ring closure from the face and others. Peck´s word of caution of 1985 was recalled as being valid whenever Interactive Design is used to “disguise poor design or excuse shoddy investigations”. Finally, the differences between ductile and brittle scenarios for tunnels in soil and the impact on Interactive Design was reviewed, with overall conditions favouring the use of Interactive Design in ductile and brittle scenarios, where potential optimizations are higher and risks are lower.

Selected Case Histories of tunnels were very briefly reviewed for further reading reference and were classified into two groups: research oriented monitoring and large routine projects. In the first group, reference was made to the Botlek Rail Tunnel in the Netherlands, where near zero effective stresses were measured in the EPB chamber sand-foam mixture and where fresh backfilling grout pressure gradients enhanced lining buoyancy, not followed by the TBM, producing longitudinal lining bending. Also in this group, reference was made to the 2nd Heinenoord Tunnel, once more in the Netherlands, that highlighted the importance of accounting the effects of the backfilling grouting on the loss of ground and where high pore pressure increases were measured in the soil ahead the TBM cutting face, due to the removal of the bentonite cake by the cutting tools. In the second group of case histories, large routine tunnel projects were referred to, in which considerable contribution was made to the practice. The first case of this group was the Toulouse Subway tunnels, where the response of a particular ground type to three very distinct TBM construction technologies was compared in a detailed field instrumentation investigation. The second case of this group was the Brasilia Metro tunnels, quoted as a successful application of an ab initio Interactive Design in a ductile environment, on which tunnel failure was preduded in a section by a design change triggered when limiting values of the performance indicators discussed herein where reached.

The basic benefits of *Geotechnical Instrumentation* were reiterated and the principles of monitoring and the principles for systematic development of a geotechnical monitoring system were reaffirmed. New advances in geotechnical monitoring were reported, focussing specifically on the fields of ground investigation, high capacity pile load testing and geotechnical monitoring using fibre optic technologies.

In the field of ground investigation, the application of Jean-Lutz parameter monitoring and the geophysical technique using borehole radar in drilling and grouting works were illustrated as probing and parameter quantification systems in highly variable dolomitic soil, and rock profile conditions where conventional means may not be suitable. In the field of high capacity pile load testing, the use of Osterberg Cell® technology was described. The ability to test piles to the order of 300 MN is an exciting prospect for design and construction of piled foundations.

Adding to the arsenal of exciting advances in geotechnical monitoring are recent advances in the development of fibre optic (FO) sensors. Most of these advances occur in the field of Structural Health Monitoring and occur across different engineering disciplines. In geotechnical engineering it is in particular the advances on the fronts of SOFO interferometric and fibre Bragg grating (FBG) point measurement systems, and Brillouin Optical Time Domain Reflectometry (BOTDR) distributed sensing systems that are of interest. Although the three systems are used in a variety of applications, their distinguishing features are highlighted in this report.

Possibly the most significant advance in FO technology is the advance in distributed sensing using BOTDR technology, which enables the monitoring of strain and temperature changes over several hundreds of kilometres. Recent advances in BOTDR technology also illustrated the successful use in dynamic applications (albeit the application frequencies to date are lower than what can be distinguished by FBG systems). BOTDR technology in terms of FO sensing is in an active development process, which makes it, of the three most prominent FO technologies discussed, the focus of new development.

In relation to point measurement systems, SOFO systems have the inherent characteristic of not being affected by local effects such as cracking, due to its long gauge length. These systems are therefore of particular value where average strain is sufficient. FBGs on the other hand are known for their ability to monitor dynamic changes in strain and temperature.

To illustrate the successful application of these new advances in geotechnical instrumentation a number of case studies are discussed. The case studies display the development of each technology over recent years and illustrate the particular strong points of each type of instrumentation. The case studies further highlight why these systems are regarded as some of the most inspiring advances in geotechnical engineering over recent years. Notwithstanding the particular uses and abilities of different technologies, it is not the objective to identify ‘the most suitable sensor of all’, but rather to illustrate that, when used at their most suitable strength, combinations of sensors provide a mighty arsenal to the practitioner in the development of effective monitoring systems. This was illustrated to good measure in the conceptual monitoring layout for an arch dam.

The field of instrumentation allows the interfacing of technologies not only within the fields of geotechnical engineering or even civil engineering, but allows cross-pollination between civil engineering, aeronautical engineering, mechanical engineering, electronic engineering and electrical engineering. Everyone is getting the benefit of a vast database of knowledge and research, which is to the advantage of all participants and users of the different technologies and provides for a platform of continuous development of further advances in instrumentation.

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