INSTRUMENTATION FOR UNDERGROUND CONSTRUCTION PROJECTS

by

Za-Chieh Moh and Richard N. Hwang

Reprinted from

Proceedings of Twelfth Southeast Asian Geotechnical Conference and the Fourth International Conference on Tropical Soils

Keynote Speech, Kuala Lumpur, Malaysia

May 6~10, 1996, Vol. 2, pp.113-129

Za-Chieh Moh, President, Moh and Associates, Inc., Taipei, Taiwan, Republic of China
Richard N. Hwang, Project Manager, GESC, Moh and Associates, Inc., Taipei, Taiwan, Republic of China

Instrumentation for Underground Construction Projects

Keynote Speech, Twelfth Southeast Asian Geotechnical Conference
6-10 May 1996, Kuala Lumpur, Malysia

SYNOPSIS: This paper discusses the various aspects of instrumentation for large projects, including functions and applications of instruments, planning and implementation of instrumentation programs, and collection, reduction and interpretation of data. Examples are given and results illustrated with the aim that instrumentation programs can be proper planned and executed in the future.

1. INTRODUCTION

For deep excavations in cities, the safety of the excavated pits and adjacent structures is always a serious concern. In fact, structural failures of temporary works resulting in serious, even fatal, damages are not uncommon. To enable the works to proceed smoothly, a comprehensive instrumentation and monitoring program is a necessity rather than luxury. It is the intention of this paper to discuss the various aspects of instrumentation, including functions and applications of instruments, planning and implementation of instrumentation programs, and collection, reduction and interpretation of data, with the aim that instrumentation programs can be properly planned and executed in the future.

Rapid transit systems are probably the largest underground projects in cities. In the Southeast Asia, following the completion of Hong Kong MTR and Singapore MRT systems, mass rapid transit systems are being constructed in Taipei and Kuala Lumpur. Bangkok is also joining the bandwagon with the engagement of Project Management Consultant soon. Although this paper uses the Taipei Rapid Transit Systems (TRTS) as an example for illustrations, the principles outlined herein are equally applicable to other rapid transit systems. The portions related to cut-and-cover constructions are applicable to any types of deep excavations.

2. BENEFITS OF INSTRUMENTATION

Naturally, every instrument installed must serve the purpose of answering a specific question, therefore, it should be selected to suit this purpose. Wrong instruments in the wrong places provide information that may at best be confusing and at worst divert attention from diagnosing signs of trouble. It is also important to understand the costs and the benefits involved and to balance the two. Too much instrumentation is wasteful, while too little, arising from a desire to save money, can be more than false economy: it can even be disastrous.

The benefits of instrumentation can be summarized as follows (Dunnicliff and Green, 1988)

(a) Benefits During Design

- Definition of initial site conditions
- Proof testing
- Fact-finding in crisis situation

(b) Benefits During construction

- Maintaining Safety
- Observational method
- Construction control
- Providing legal protection
- Measurement of fill quantities
- Enhancing public relations
- Advancing the State-of-the-Art

(c) Benefits after construction is completed

- Maintaining Long Term Safety

It has been fully appreciated by engineers that soil engineering, being extremely uncertain in nature, is an art rather than science. Judgment plays a by far dominating role than calculations. In other words, we learn from past experience and design procedures are constantly under revisions based on experience gained. Instrumentation is the only way for obtaining quantified experience. In his Ninth Rankine Lecture, Prof. Peck set out procedures for the observational method as applied to soil mechanics (Peck, 1969). The method provides a way of controlling safety while minimizing construction costs, so long as the design can be modified during construction. Peck's observational method involves developing an initial design based on most probable conditions, together with predictions of behavior. Calculations based on most unfavorable conditions are also made and these are used to identify contingency plans and trigger values for the monitoring system. Construction work should be started using the most probable design. If the monitoring records exceed the predicted behavior, then the pre-defined contingency plans would be triggered. The response time for monitoring and implementation of the contingency plan must be appropriate to control the work. In fact, these principles had long been followed before Professor Peck laid them out and all the works involving soils are based on the observational method, either implicitly or explicitly.

3. PROJECT MANAGEMENT

If an instrumentation program is to be successful, it has to be planned by experienced geotechnical engineers and executed by experienced specialist subcontractors. More importantly, it has to be supervised by experienced geotechnical engineers, preferably the same ones who laid out the program, either as staff of the supervising team or as an independent third party. What has happened in many large projects was that instrumentation programs were planned by designers who would not be involved in construction or had little authority in enforcing the programs to be executed in the way they were planned. In such a way the essence of the programs was difficult to retain. There is absolutely no incentive for contractors, whose major concerns are cost and progress, to do a good job on instrumentation, and more than frequently, instruments are considered by contractors to be obstacles to construction. The purposes of instrumentation will not be able to achieve if a program is totally left to the contractor without proper supervision.

For the TRTS Project, for example, the importance of geotechnical engineering has been well appreciated by the project owner, the Department of Rapid Transit Systems (DORTS), and a team of geotechnical engineers has been retained as Geotechnical Engineering Specialty Consultant (GESC) throughout the various stages of the Project. Figure 1 shows the organization chart of GESC. At the time this paper was prepared, GESC has been involved in a total of 48 civil structural construction contracts. Typically, each contract covers one to two stations and viaducts/tunnels linking these stations. In the design stages, GESC is responsible for reviewing the designs and the instrumentation programs set by

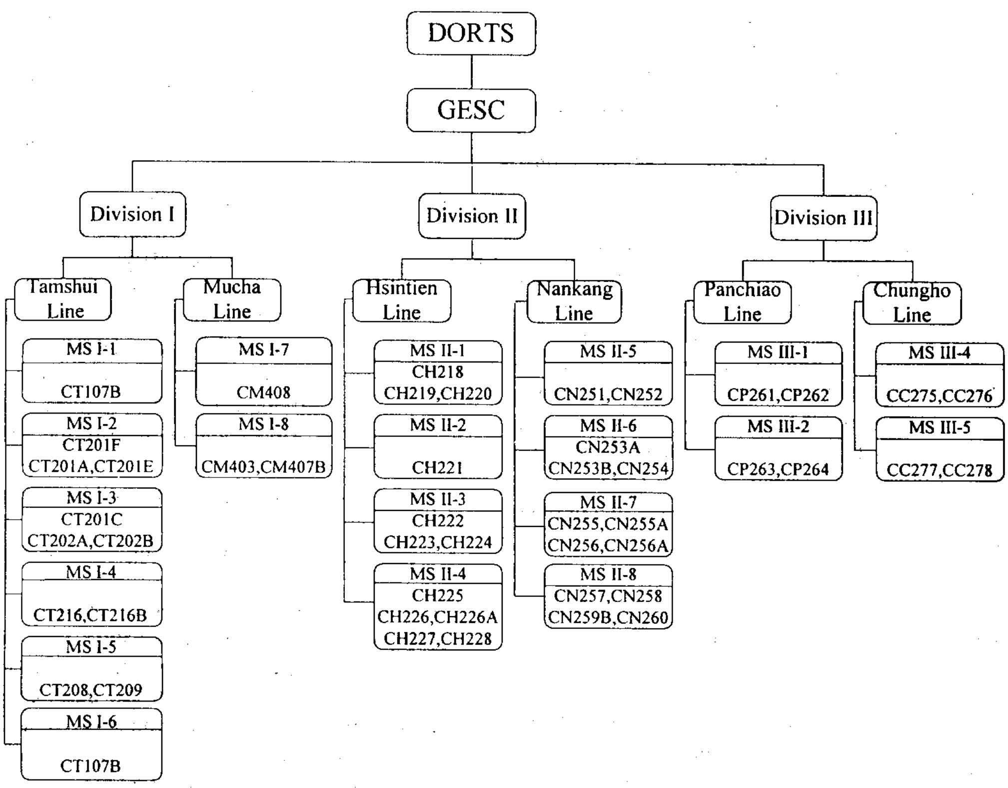


Fig. 1 Organization Chart for GESC, TRTS Project

the Detailed Design Consultants (DDC) engaged by DORTS. During constructions, GESC provides advices to the supervising staff of DORTS and is assigned the duty of compiling instrument readings taken by the contractors.

Each contractor is required by the contract to retain a qualified geotechnical engineer full time at the site. This helps to maintain the quality of works in general. To enable GESC to have first-hand information, 20 monitoring stations have been setup at the sites. The contractors are responsible for taking instrument readings and transmitting readings in a pre-specified format to these monitoring stations. As illustrated in Fig. 2, data are first checked by the GESC staff at the monitoring stations. Contractors are required to verify questionable data, provide explanations, or confirm them at sites if necessary. As part of quality assurance program, GESC also takes selective readings independently, or jointly with the contractors, to ensure that readings are accurate and reliable.

Top priority has been assigned to the protection of structures adjacent to the works at all the stages. Based on the conditions of these structures, DDC set out criteria for contractors to follow. During constructions, to ensure that actions can be taken in time, notices will be served by the Monitoring Stations as follows:

(a) Warning - abnormal phenomenon has been observed or the current reading has reached a half of the alert level. The supervising staff and the detailed designer are notified for potential risks. The contractor is requested to review his construction method and construction sequence to see if something is going wrong.

(b) Alert - current reading has reached the pre-specified alert level. The contractor is requested to explain what has happened and propose contingency measures to be reviewed by the supervising staff and the detailed designer.

(c) Action - current reading has reached the pre-specified action level. The contractor is requested to execute the contingency plan which has been reviewed and approved. In the meanwhile, the District Project Office and the Quality Assurance Officer are notified. They will follow up the event till the risk is over.

The days are long gone for manual processing of instrument readings. Every type of instrument involves a certain degree of computerization at one stage or another. For TRTS Project, up to January 1996 when this paper was prepared, more than 22,000 pieces of instruments of various types have been installed and the quantity of readings has exceeded 10 million sets. It is simply impossible to handle this enormous amount of data manually. To ensure that warnings are received timely in case there is a potential danger, instrument readings are compared with pre-specified warning levels systematically using a software package developed by GESC as soon as they are received by the Monitoring Stations. If any of the criteria is exceeded, a warning notice is automatically issued to concerned parties. Through a fax-modem, the notices can either be sent to a fax machine or to a computer network for distribution.

Notwithstanding all the precautions, accidents did happen. GESC assisted the supervising staff in identifying the causes of accidents and actively participated in all phases of works to ensure that the remedial measures were effective and were taken timely.

Instrument data are transmitted by Monitoring Stations to the Integrated Data Storage Center (IDSC) managed by GESC for safekeeping and for analyses. If it is found that the current design is deficient or the design assumptions are not met, the designers and the contractors will be informed for the designs to be revised. GESC also selects clusters of instruments for researches and make necessary efforts in ensuring that sufficient high quality data are obtained. In the period between 1987 and 1995, more than 50 technical papers covering all aspects of geotechnical engineering were published. This enables design procedures to be calibrated by local experience.

The budget for the GESC equals to roughly 0.6% of the total cost of civil contracts. Because of the difficult ground conditions along the routes and the lack of local tunnelling experience, this amount is deemed to be appropriate and necessary. The spending has been well paid off. The quantifiable savings alone exceed the costs by at least a couple of times through the optimization of designs provided by the GESC. The costs of instrumentation, including installation and monitoring by contractors, amount to about 0.7% of the total cost of civil contracts. This percentage is comparable to the percentages for projects of a similar nature worldwide.

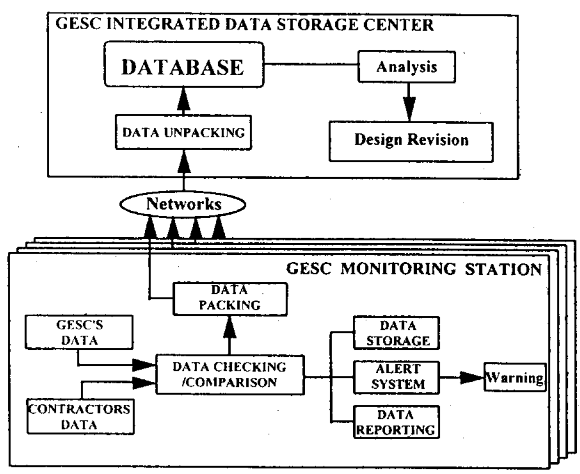


Fig. 2 Data Collection, Verification and Transmission

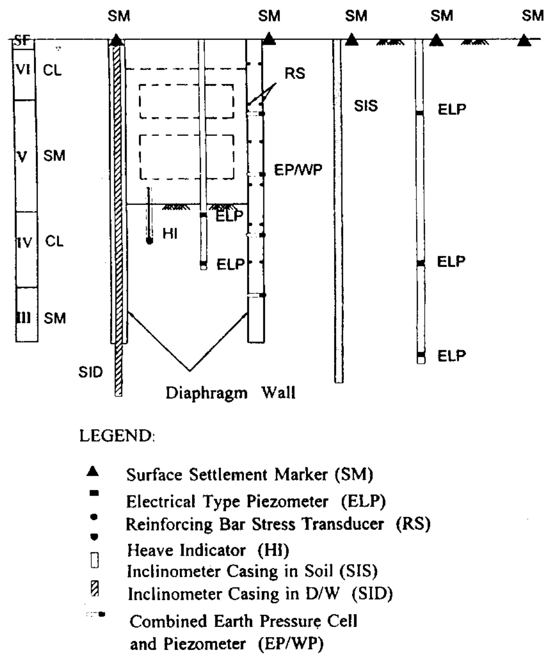


Fig. 3 Typical Instrumentation for Cut-and-Cover Constructions

4. CUT-AND-COVER CONSTRUCTIONS

Figure 3 is a schematic instrumentation program for a typical excavation using the cut-and-cover method. The types of instruments commonly adopted, recommended practice, examples and results obtained are given as follows.

4.1 Strut Loads

Strut load is probably the most important item to be monitored because it is a major safety concern. Failures of struts are not uncommon. They are not necessarily a result of defective design but may also be a result of negligence. Their consequences are usually serious and sometimes may even be fatal. With the recent development on computers, engineers nowadays rely more and more on softwares for designing retaining systems. There is a growing danger of using such softwares without calibrations for local experience. It is the authors' experience that, while computer analyses provide reasonable bending moments and shear forces in walls, strut loads obtained in computer analyses are quite doubtful. This is because of the fact that strut loads are affected by construction sequence, promptness of strutting, workmanship, and many environmental factors which are beyond the control of the designer. Therefore, monitoring of strut loads is crucial for deep excavations. In areas without much past experience or areas with complex ground conditions, as a general rule, 15% to 20% of struts shall be instrumented as a precaution. Even in areas with sufficient experience, an absolute minimum percentage of 10% is recommended. Instruments should concentrate in selected sections to facilitate back analyses. For large excavations carried out in phases, more instruments should be installed in the early phases, rather than in the latter ones, to gain experience so the remaining strutting system can be optimized.

Preloading of struts has been found useful in reducing ground movements and is getting popular in cities because of the concern of damaging structures adjacent to the works. Frequently, strut loads are monitored by using loading jacks used for preloading struts. Loading jacks are not well sealed and even minor leakage will lead to relaxation of loads. Load measurements based on jack fluid pressure may be significantly in error, may be on the unsafe side and are usually unacceptable (Dunnicliff and Green, 1988). Figure 4 shows the results of preloading of a strut specifically instrumented for research purposes. The strut was first loaded to the design load of 40 tons. After the pre-specified load was reached, the gap between the two sections of the strut was wedged by steel plates and the oil pressure in the jack was released for the intention of removing the jack for reuse. However, it was observed that the load in the strut was totally lost. The strut was reloaded to 40 tons and the gap was wedged by a piece of H-section steel cut to the size of the gap. The oil pressure was released for the second time and the load dropped from 40 tons to 25 tons. This was unacceptable and the strut was then loaded again to 82 tones. Steel plates were forced into the gap between the strut and the H-section relieving part of the load in the jack and, meanwhile, increasing the load in the strut as sensed by load cells (LC1 and LC13) and strain gauges (VG3, 4, 9 and 10). The load dropped to 40 tons after the jack was removed and stayed at that level subsequently. This example illustrates how sensitive could strut loads be to construction details.

Figure 5 shows the loads measured by load cells (LC) and vibrating-type strain gauges (VG) mounted on the said strut in the first cycle of loading as compared with the readings on the loading jack. As can be noted, the differences are as much as

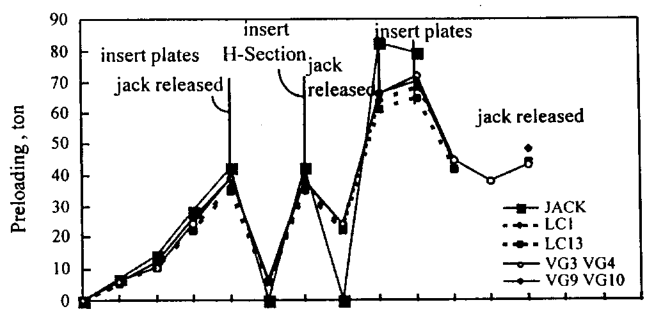


Fig. 4 Pre-loading A Strut

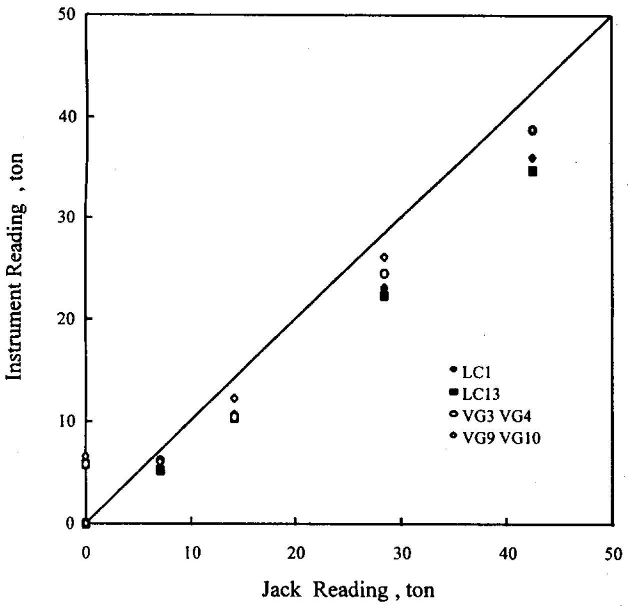


Fig. 5 Loads Measured by Different Devices during Preloading

15%. At the peak load of 82 tones in the third cycle, LC13 showed a load of 61 tons, giving a difference of 25%. All these devices were carefully calibrated beforehand and were carefully installed for research purposes. The differences would have been larger otherwise.

Fellenius (1980, 1984) reported on the use of a load cell and a calibrated hydraulic jack during pile load testing and indicated that during loading the jack typically over-registered by 10-25%, and during unloading it typically under-registered by 5%. This finding is in agreement with the above experience.

In most of cases, loading jacks were not removed and the loads were locked off by using the locking-off devices equipped on the jacks. In such cases, the release of oil pressure in jacks still caused losses of about 15% of the pre-loads. Therefore, if the design pre-loads are to be maintained, an extra of 15% should be applied. The reuse of loading jacks may not be economical because much larger sections are required to sustain the extra pre-loads.

Loads in a strut are affected by too many factors. Temperature, for example, is one of them. It is therefore recommended to take readings when temperature is steady, for example, before 10am or after 4pm. Misalignment is a common problem resulting in bending in struts. Therefore strain gauges are always installed in pairs, one on each side of the web of a H-beam. Theoretically, the average of readings of two strain gauges will supposedly provide reliable axial loads. This will be the case if and only if both gauges function properly. If either of them is not installed properly or malfunctions, the true strut load will never be able to obtain. Increasing the number of gauges in a section does not necessarily help. The authors have the experience that the results from six strain gauges, two on the web and four at the ends of the flanges, could not be meaningfully interpreted. It was very difficult to figure out which of the gauges was not giving correct readings. Impacts of construction machines or tools on struts, which occur frequently but are seldom reported, may result in totally misleading readings and lead engineers astray.

It is suggested, as a routine, to compute apparent earth pressures from the measured strut loads in each stage of excavation and compare these pressures with the Peck's diagram (Peck, 1969a), refer to Fig. 6, which has been used for decades and is still specified in many design codes and manuals. For stiff ground with $\gamma H/Cu \leq 4$, where γ = unit weight of soil, H = depth of excavation, Cu = undrained shear strength of soil at base, the pressure envelope obtained from observed strut loads should be in a reasonable agreement with the Peck's diagram. If not, confirmation of readings is necessary. After all, the Peck's diagram has been proved to provide reasonable results in numerous cases and has been supported by many technical papers and reports. For soft ground with $\gamma H/Cu > 4$, however, strut loads are affected by toe stability and the Peck's diagrams may not be relied upon.

Diagonal struts at corners of excavated pits are often overlooked. There was an accident recently in Taipei which was attributed to the shearing failure of bolts on diagonal struts. Although earthpressures are expected to be less at corners because of the three-dimensional effects, the balance of loads at corners is complicated, particularly when diagonal struts are not at the same levels with other struts. Therefore, attention should be paid to structural details.

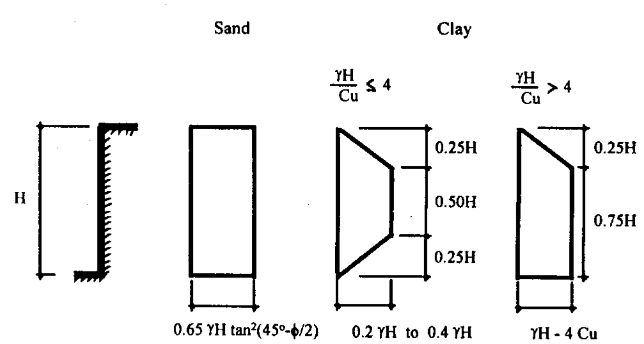


Fig. 6 Peck's Diagrams for Apparent Earth Pressures

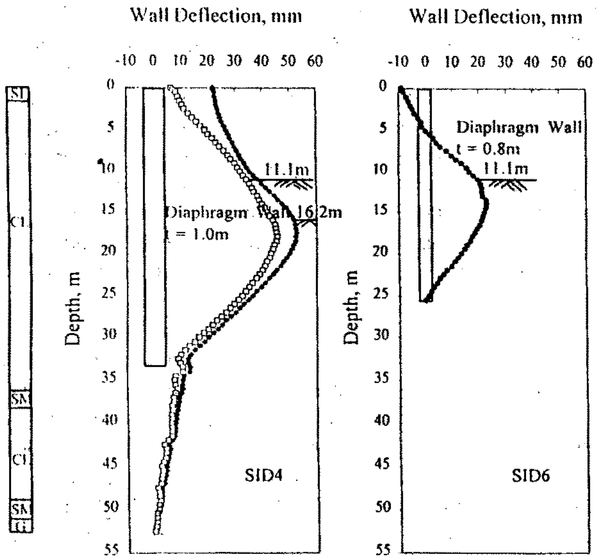


Fig. 7 Effects of Embedment on Inclinometer Readings

4.2 Lateral Deflections of Walls

Lateral deflection of retaining structure is one of the issues attracting the most attention because it is directly related to ground settlement which is a major source of damages to adjacent buildings. The observed lateral deflections can also be used to back-calculate bending moments and stresses in flexible steel structures such as sheet pile or steel piles. However, it is cautioned that such calculations may not be relied on for very rigid walls of which deflections are small or for walls of which the section moduli are difficult to determine (Wolosick and Feldman, 1987; Gould and Dunnicliff, 1971; Saxena, 1974; Soares, 1983). The authors also have attempted for several cases to back-calculate bending moments from inclinometer readings and could not obtain consistent results.

It is still popular to determine the penetrations of retaining walls based on equilibrium of ultimate active forces and passive forces on the two sides with a proper factor of safety. It should be noted that a certain wall movement is required for passive resistance to develop. It is uneconomical to purposely limit the toe movement of a diaphragm wall to a minimal. However, excessive lateral movement of wall below the bottom of excavation is a fore-warning of failure which may be a result of toe instability or base heave. Both events are more likely to occur in soft clays than in sands.

Inclinometers are recommended to be installed at 30m to 50m intervals. For a typical underground subway station of, say, 200m to 250m in length and 20m to 30m in width, four to five inclinometers are recommended along each longside and one inclinometer at each end. The tips of inclinometers are usually assumed to be fixed with zero movement and are used as a reference points. It is therefore necessary to install inclinometers all the way down to the underlying hardpan. However, sometimes the hardpan is located at a great depth and it is impractical to extend inclinometers to that depth. In such cases, it is necessary to measure the absolute displacement at the top and calibrate the lateral displacements at all other depths. This procedure is seldom followed because at site it is not easy to measure the displacement at the top to the accuracy comparable to the desired accuracy for inclinometers.

Figure 7 shows a comparison of the deflections of two inclinometers installed in one of the contracts of the Nankang Line. The site is located in the K1 Zone of Taipei Basin. Because of different depths of excavation of 11.1m and 16.2m, the diaphragm walls have different thickness, i.e., 0.8m and 1m, and different penetrations, i.e., 26m and 33m, respectively. Inclinometer SID4 was installed all the way down to the gravelly hardpan at a depth of 53m below surface and Inclinometer SID6 was installed to a depth of 26m only. At the final stage, the deflected shape of the upper 26m of SID4 was almost identical to that of SID6. At the depth of 26m, however, SID4 showed an inward movement of 32mm while SID6 was assumed to be unmoved. It is conceivable that the toe of SID6 indeed moved by 32mm or so laterally and the top of SID6 should have an inward movement of 20mm, as was the case for SID4, instead of an outward movement of 10mm as reported.

In some of the contracts, one-third of inclinometers were anchored in the hardpan and the displacements at the toe level of diaphragm wall were used to calibrate the displacements obtained by other inclinometers which stopped short. Sometimes, toe movements can be observed from inclinometer readings even without referring to other inclinometers. If the bottom segment starts to deviate from the vertical, as was the case for SID6 in Fig. 7, the toe is likely to start to move. However, the fact that the bottom segment remains vertical does not necessarily guarantee the fixture of the toe. The entire segment may move horizontally without tilting.

The delay in strutting after excavation may add much to the lateral deflection of a wall, particularly in clays. The problem is more serious when excavation and strutting are executed by two different sub-contractors. Each has his own priority and wall movement is a concern to neither of them. In one project the authors have been associated with, excavation was carried out to the fifth strutting level while there were only two levels of struts in place. Although the diaphragm wall did not collapse, the excess inward movement necessitated the modification of the structural design of the inner wall.

Figure 8 shows an interesting case in which the sandstone bedrock underlying the soft deposit is inclined with a differential depth of 20m between the two sides. Excavation was carried out to a depth of 23.3m and six levels of struts were used. When the

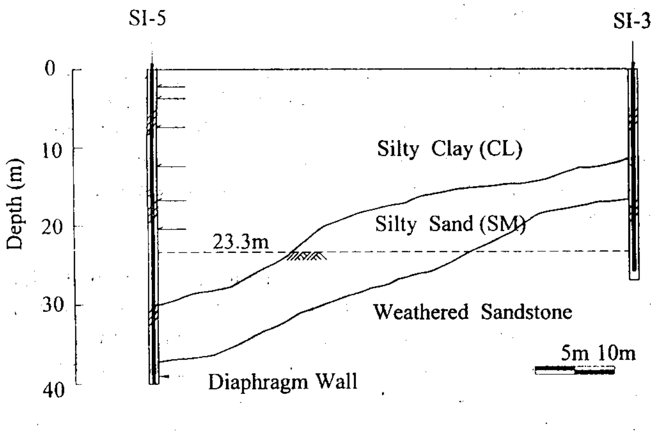


Fig. 8 A Site with Sloping Bedrock

first level struts were installed, the deflections of the two inclinometers installed in diaphragm walls on the two sides were practically the same with maximum deflections of about 40mm at the top. However, at the final stage, Inclinometer SI5 on the side with a deeper depth to the bedrock, refer to Fig. 9, bulged out to give a maximum lateral deflection of 200mm which occurred at the level of the bottom of excavation while Inclinometer SI3 on the other side with a shallower depth to the bedrock, refer to Fig. 10, was pushed outward with a net outward movement of a few millimeters at the top . This was obviously a result of imbalance of pressures on the two sides. Such interaction effects have seldom been taken into account in designs and are difficult to estimate even someone wishes to.

Retaining walls may also move outward as a result of excessive pre-loads. The outward movements of a rigid wall due to preloading of a strut are likely to reduce loads in adjacent struts. It has also frequently been observed that in soft clays, the upper portion of diaphragm wall has the tendency of moving outward as the lower portion of the wall bulges .

4.3 Settlements of Walls

Settlements of retaining walls are often overlooked. As excavation proceeds, refer to Fig. 11, the ground on the active side (i.e., outside) tends to drag the wall down. The downdrag and the self weight of the wall are resisted by the positive skin friction on the passive side (i.e., inside) of the wall and the resistance at the toe. For diaphragm walls, sludge is usually present, no matter how much effort has been spent in cleaning, at the toes and meaningful resistance will not develop till a certain amount of settlement has occurred. Furthermore, as more and more soil is removed, the less and less the embedment will be. It is therefore not surprising for diaphragm walls to undergo settlements of a few millimeters to a couple of centimeters.

Wall settlements of a couple of centimeters are usually inconsequential. However, in a few accidents as a result of piping leading to undermining of the toes, it was suspected that large settlements of diaphragm walls had occurred. Unfortunately data were unavailable for estimating the actual amounts of settlements. It is therefore suggested that the top levels of retaining walls be taken at, say, 20m intervals and documented as a record. The efforts are really minimal and the information will be useful in arresting arguments.

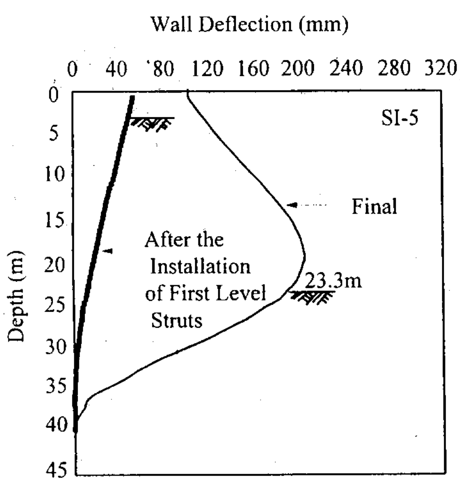


Fig. 9 Movements of Wall Measured by Inclinometer SI5

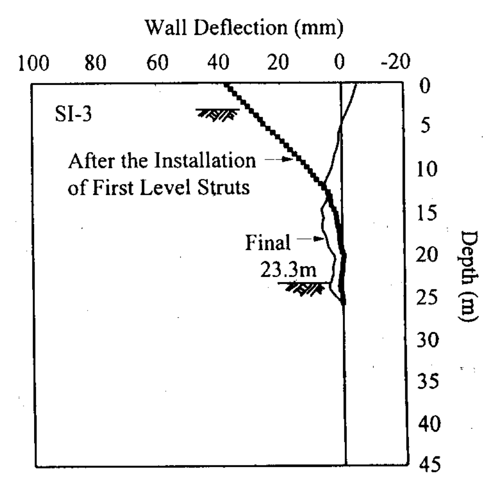


Fig. 10 Movements of Wall Measured by Inclinometer SI3

4.4 Stresses in Reinforcements

Stresses induced in sheet piles are seldom monitored because of the difficulty in protecting gauges during driving of sheet piles. For diaphragm walls, strain gauges are commonly used to monitor the stresses induced in reinforcements. The exercise is more or less an academic one because design tends to be conservation and rupture of wall members as a result of bending seldom occur, at least not to the authors' experience. Yielding of rebars, if does occur, will simply lead to cracks of concrete and large lateral deflections of the wall and will not be fatal.

A good agreement between the stresses measured by strain gauges and the predicted stresses usually can not be expected because of the difficulty in estimating the section modulus of the wall. For this reason, the value of such measurements is quite limited.

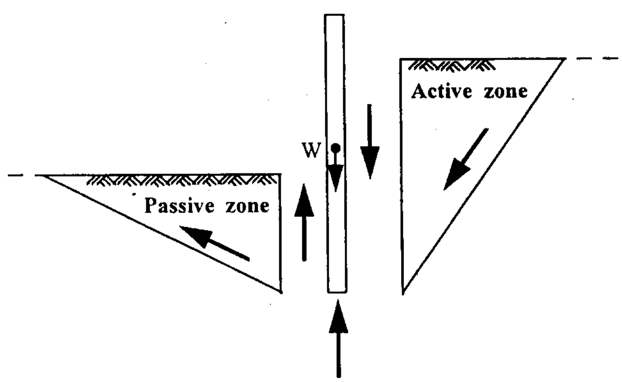


Fig. 11 Forces Acting on A Retaining Walls

4.5 Ground Heaves

In soft clays, ground heave as a result of stress relief is a common phenomenon and should be estimated beforehand. Heaves are monitored by heave rods which are shortened section by section as excavation proceeds. The rate of heave is an index of problem, if any. If the rate increases suddenly, a surcharge load (usually sand bags or construction materials) should be placed on the bottom of excavation immediately. If situation becomes critical, the pit should be backfilled or flooded as soon as possible.

The survival rate of heave rods is pretty low because rods are very easy to be knocked down by excavators. Therefore, one should be more generous in estimating the quantity of the rods to be installed in preparing the instrumentation program.

4.6 Ground Settlements

Monitoring of surface settlements is routinely carried out for all deep excavations. For this reason, studies are numerous and the results have extensively been reported. It is not intended to present any examples nor to summarize any findings herein. Instead, a few problems associated with the measurements are discussed.

Most of the errors in settlement readings are associated with unstable bench marks. Permanent bench marks must be placed on hard ground or on a rigid structure with a stable foundation outside the zone of influence of excavation; and must be verified periodically by checking with bench marks of higher precision. In some cases, it may be more economical to install a deep settlement gauge to the underlying hardpan at the site if a suitable object is unavailable nearby. However, it should be noted that the casing which is difficult to be removed, may become an obstacle for future underground works.

Taking settlement readings is not a easy job in congested cities. In the first place, traffic vibration, heat waves and exhaust from vehicles make readings unstable. Secondly, settlement markers installed on pavements are often damaged by vehicles and re-installation always lead to the question of how much settlement has been missed during the period. Adjustment of readings is not as easy as one may assume when many parties are involved, particularly when stringent criteria are specified. Re-paving of road surface frequently necessitates the replacement of a large number of markers. Contractors always argue that some of the settlements were due to contraction of new asphalt and should be discounted. One way to reduce the arguments is to install markers at a certain depth, usually 1m to 1.5m, below surface, rather than at the surface.

4.7 Ground Water Levels

More than a half of accidents in underground works are related to water. Leakage at diaphragm wall joints is a common phenomenon. Long-term leakage may lead to sinkholes resulting in rupture of utilities. The consequences could be very serious and sometimes could even be fatal. Unfortunately most of contractors are optimists always hoping for the best. They are willing to spend the minimum efforts in preventing problems to occur. Grouting is usually carried out when the problems are already serious and halts once water stops flowing. It is very difficult to ask contractors to probe the ground to see if there are cavities resulted from erosion of soil particles.

Piezometers are not very useful in pinpointing leakages because readings are affected by too many factors. Rains and droughts are obviously a major reason for groundwater level to rise or to fall. In coastal areas or in alluvial fans, the groundwater levels tend to fluctuate with tides or with the water levels in rivers. The fact that piezometer readings are steady does not guarantee the absence of problems. It simply means the balance of rate of replenishment and the rate of leakage.

In soft/compressible ground, lowering of groundwater levels outside pits may lead to considerable settlements which may become detrimental to adjacent buildings and should be avoided. Inside the pits, groundwater should be maintained at a level of 1m below the bottom of excavation for workability, or at a level necessary to maintain an adequate factor of safety against blow-in or piping. If the calculated factor of safety against blow-in or piping is insufficient or marginal, water levels should be lowered and twice as many piezometers as needed must be installed for ensuring that piezometric heads do not exceed the critical values because, with good luck, only a half of them will survive. Replacement of damaged piezometer when excavation is already deep is a dangerous operation and should be executed with great care. It has happened in quite a few incidents that drilling through clay blanket for installing wellpoints or piezometers created water fountains which could only be stopped by flooding the pits .

Full saturation of the tips is absolute necessary for piezometers to function properly and this may not be understood by an inexperienced worker. In sandy layers standpipes can be used, but in clayey layers sensitive piezometers should be used for quicker response. Boreholes should be properly sealed, except for a short section where the piezometer tip is located, so it will not become a water path.

4.8 Earthpressures on Walls

Earthpressure is a vital piece of information required in design of retaining walls and is of great interest to designers. However, earthpressure is probably the most difficult thing to measure because of soil arching effects. For this reason, reliable data are extremely rare. It is very important to make sure cells are functioning and obtain reliable readings during installation. It is also important to apply an initial pressure equivalent to the overburden pressure, or to a pressure with a coefficient of earthpressure of 0.6 to 1. This is important for cells on both sides of the wall because earthpressures, as to be illustrated, on both sides drop as excavation proceeds. It has happened in some cases that the initial pressures were too low and the purpose of monitoring ground response was totally defeated.

Figure 12 shows the soil profile for CKS Memorial Hall Station of the Hsintien Line (Hwang, Liao and Fan, 1996). The excavation was carried out to a depth of 23.4m in the T2 Zone of Taipei Basin. Also shown in the figure are the depths at which earthpressure cells were installed. The progress of excavation is given in Fig. 13 and the variation of earthpressures on the active side of the diaphragm wall obtained by Cell EP11, installed at a depth of 28.7m, is shown in Fig. 14. Usually, each total earthpressure cell is paired with a piezometer so the effective pressure can be obtained by subtracting the pore pressure reading from the total earth pressure reading. As can be noted from the figure that the pore pressure readings, u, dropped by as much as

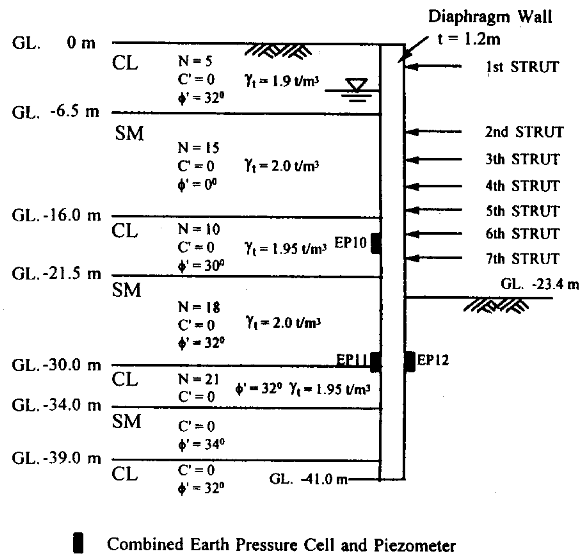


Fig. 12 Profile for CKS Memorial Hall Station of Contract CH219

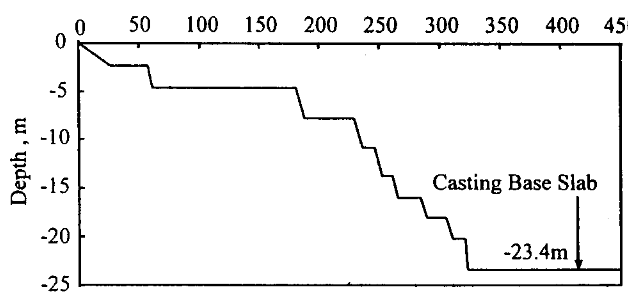


Fig. 13 Progress of Excavation, CKS Memorial Hall Station

10m at the final stage. This might or might not be related to the lowering of ground water table. Dilation of soil mass in response to wall movement may also be responsible for the reduction in pore water pressures. The total horizontal pressures, σ_h, dropped by an even greater magnitude. Therefore, the effective horizontal pressures, σ'_h, as can be noted from the figure, decreased as excavation proceeded. The overburden pressure, γH, remained unchanged, therefore, the ratio of $\sigma'_h / \gamma' H = \sigma'_h / (\gamma H - u)$ decreased from the initial value of 0.65 to 0.4. The relationships between this ratio and wall movement obtained by all the cells installed at this site is shown in Fig. 15. The data points with hollow symbols are for cells installed above the bottom of excavation and were more likely to be affected by preloading. If these points are ignored, the trend for the ratio of $\sigma'_h / \gamma' H$ to decrease with wall movement becomes clearer.

The variation of passive earth pressures obtained by Cell EP12 is shown in Fig. 16. The total horizontal earth pressures were steady till an elapse time of 200 days when excavation reached a depth of 8m and then dropped gradually. However, since the overburden pressure, γH, decreased at a faster rate, the ratio of $\sigma'_h/\gamma' H$ in fact increased till an elapse time of 260 day when the excavation reached a depth of 16m. It stayed in the range of 1.5 to 1.7 subsequently, except that a couple of readings indicated higher values. Unlike the case for the active pressures, the ratio of $\sigma'_h/\gamma' H$ for passive pressures can not be related to wall movements because there were four events occurring at the same time, i.e., (a) excavation reducing γH, (b) forced dewatering reducing u, (c) inward movement of wall and (d) settlement of wall with unknown effects.

5. SHIELD TUNNELLING

Tunnels are being built as sewers, expressways, and passageways linking stations of rapid transit systems, etc. They may be constructed either by using the cut-and-cover method or a variety of tunnelling methods, depending on costs and environmental factors. Cut-and-cover constructions for tunnels are essentially the same as the cut-and-cover constructions for other types of structures and have been discussed in the preceding section. Because of the lack of space, this section shall only address to shield tunnelling which is gaining its

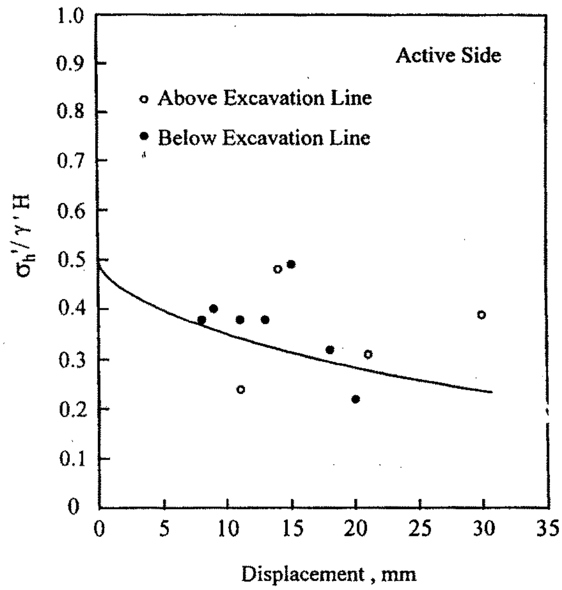


Fig. 15 Active Earth Pressures versus Displacement

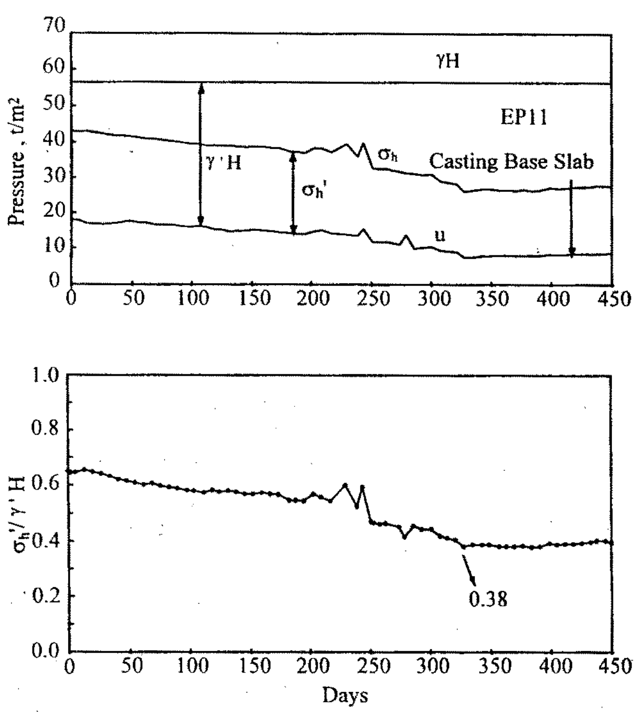


Fig. 14 Active Earth Pressures by EP11

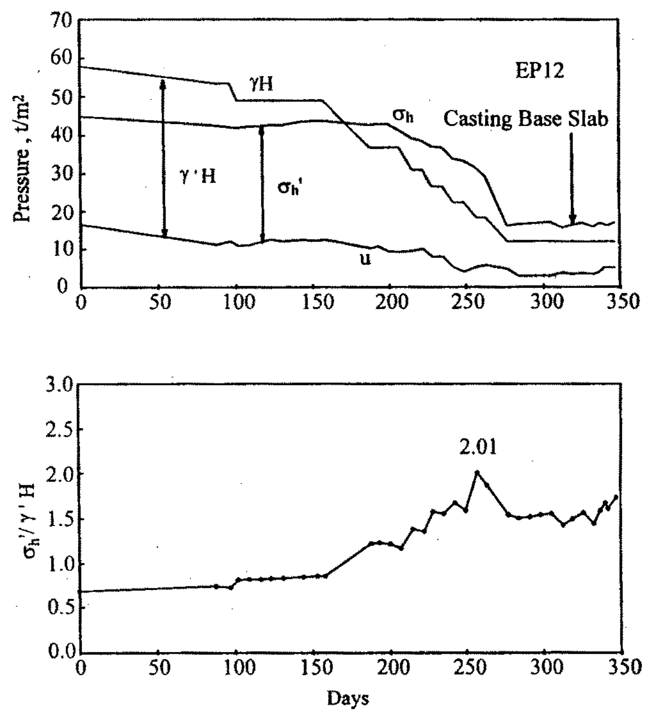


Fig. 16 Passive Earth Pressures by EP12

popularity as cities become more and more congested and tunnelling technology becomes more and more advanced.

5.1 Ground Settlements

The purpose of monitoring ground settlements over tunnels is to determine settlement troughs and to estimate ground losses. Excessive settlements may be harmful to buildings above or adjacent to tunnels. Even for tunnels passing underneath public roads, settlements of utility is of concern. Of course, the information obtained is also useful in understanding how ground responds to tunnelling so improvements can be made to reduce settlements.

It is generally accepted that the troughs of settlements over single tunnels have a shape of normal distribution as shown in Fig. 17. Settlement markers are usually installed at 5m spacings in the transverse direction of a tunnel. To have the trough reasonably covered, the instrumented section should have a width of $4i$, $2i$ on each side of center, where i is the distance to the point of inflection. For the purpose of estimating the widths of troughs beforehand, it can be assumed that $i = 0.7\ z$ where z = depth to the center of tunnel.

For twin tunnels, settlement troughs overlap as illustrated in Fig. 18. For rapid transit systems, usually tunnels are 6m in diameter and are more than 12m apart, center to center, while roads are 20m to 35m in width. It will thus be difficult to have enough settlement markers to cover the entire combined trough. However, wherever possible, for example, at intersection of two roads, it is recommended to install settlement markers to a distance of $2i$ beyond the outer edge of tunnels.

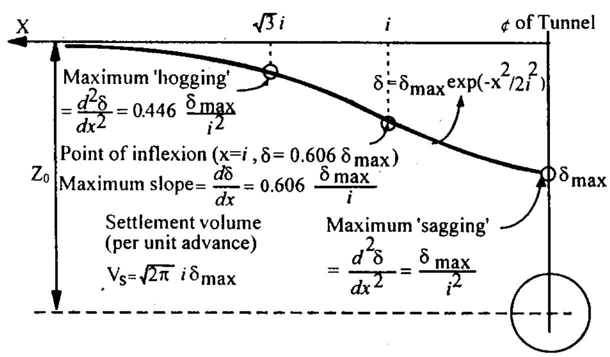


Fig. 17 Settlement Trough for Single tunnels

Studies on tunnel-tunnel interaction effects on settlements are many, however, the only conclusion which can be made is that, if tunnels are apart by a distance of twice the diameter, center to center, the interaction effects can be ignored. Closer than that, results are inconclusive. For the case shown in Fig. 18, the two tunnels are sufficiently apart and superposition of settlement troughs appears to be valid. The volumes of the settlement troughs correspond to ground losses equal to 0.5% to 1.1% of the excavated soil.

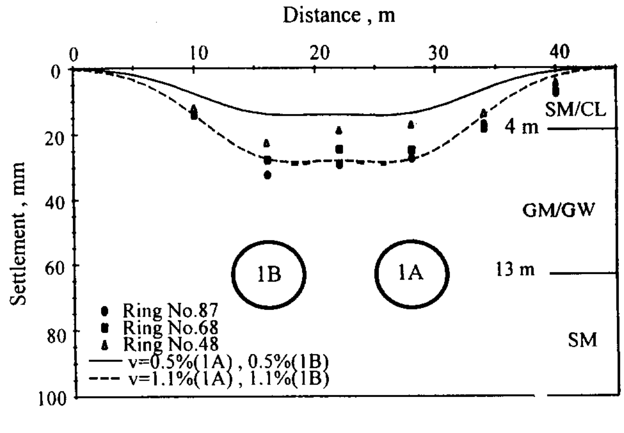
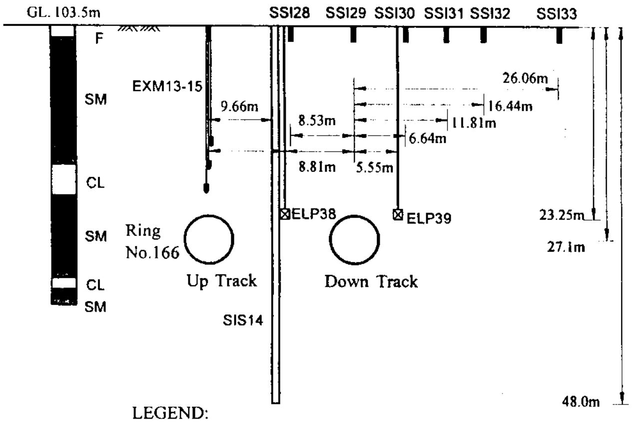


Fig. 18 Settlement Trough over Twin Tunnels

To determine settlements at various depths below surface, rod extensometers can be used (Moh, Ju and Hwang, 1996). Figure 19 shows the instrument layout and soil profile for Section T2 of Contract CP261 of the Panchiao Line and Fig. 20 shows the records of ground settlements obtained during the driving of the Up-track tunnel. As can be noted that settlements increased with depth. The tail of the shield machine passed this instrumented section roughly at 13:00 on June 25 and grouting was carried out a couple of hours later. During this period, readings were taken at 15-min intervals. This enables excellent records to be obtained. Ground heaves of a few millimeters were observed, presumably, as a result of grouting. This point will be further elaborated in the following section.

Instrumented sections are recommended to be placed at 50m to 100m intervals along the route. Initial readings should be established a week before the arrival of the shield machine. Earlier readings are not recommended because of the difficulty in safekeeping the markers. For establishing settlement trough and computing ground loss, readings are recommended to be taken daily in the 6-day period during the passing of the shield, i.e., two days before the arrival of the shield and four days after the passing of the shield, then weekly afterward. For sections specially instrumented with extensometers, inclinometers and piezometers for studying ground response to the passing of the shield, back grouting of tail void, and secondary injection, etc., readings should be taken hourly, or even more frequently, in the



LEGEND:

- ⊠ Electrical Type Piezometer (ELP)
- ▌ Shallow Surface Settlement Indicator (SSI)
- ▮ Multiple Extensometer (EXM)
- ▯ Inclinometer Casing in Soil (SIS)

Fig. 19 Soil Profile and Instrument Layout at Section 261T2

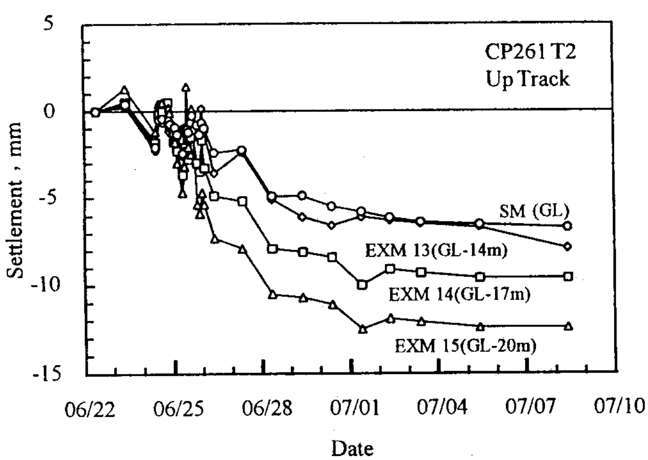


Fig. 20 Extensometer Readings at Section 261T2

afore-mentioned 6-day period. In such a case a data logger will be very useful.

In-between instrumented sections, single settlement points are recommended to be placed at 25m intervals along the alignment of each tunnel. In general, readings should be taken daily in the afore-mentioned 6-day period. Subsequent readings can be taken weekly or even monthly.

Experience indicates that in clays consolidation settlements may last for 3 months or even longer, therefore, it is recommended to take readings for at least 3 months.

It has happened in two separate occasions that concrete pavements were so rigid that surface settlement markers did not show any abnormal settlements while there were large cavities above the shields resulted from, presumably, imbalance of the volume of soils excavated and the volume of soils removed. The cavities were detected by settlement points embedded at a depth of 1.5m or so. They were promptly backfilled and potential accidents were avoided.

5.2 Groundwater Levels

Lowering of groundwater levels may induce consolidation settlements and the discussions given in Section 4.7 are valid. For tunnels driven by using open-type shield machines, or tunnels driven without using shield machines, groundwater levels should be monitored along the route for determine stability of the ground at tunnel face. This is particularly true for sandy ground for which ingress of ground water is a major source of problems. Compressed air should be applied if necessary and the air pressure to be adopted will depend on groundwater level. For closed-type shield machines, face stability is unrelated to groundwater levels.

5.3 Pore Pressure Responses

Monitoring of pore pressure response to tunnelling is a purely academic exercise. The mere purpose is to identify factors affecting long-term consolidation settlements. Observations indicate that most of excess pore pressures were induced as a result of grouting for filling up the void at the tail of the shield, rather than the advancement of shield.

In sands, because pore pressures dissipate rather quickly, readings should be taken at intervals not longer than 15 minutes, otherwise, it will not be able to catch the peak response. For this reason, a data logger will be needed and electric type piezometers will be more appropriate. It is suggested that monitoring start two days before the arrival and four days after the passing of the shield. For the case shown in Fig. 19, piezometer readings are given in Fig. 21. An excess pore pressure of 40 kPa was recorded by piezometer, ELP 38, located at one diameter away from the edge of the tunnel. The fact that pore pressures were induced as a result of grouting is evident in Fig. 22 which correlates readings with tunnelling activities (Moh, Hulme and Hwang, 1995; Hwang, Moh and Chen, 1996). It is also evident from this figure that heaves experienced by the extensometers were due to grouting. However, since extensometer readings were taken hourly, instead of every 15-minute, peak responses could have been missed in-between readings.

In clays, much longer time intervals, say, hourly, are allowed.

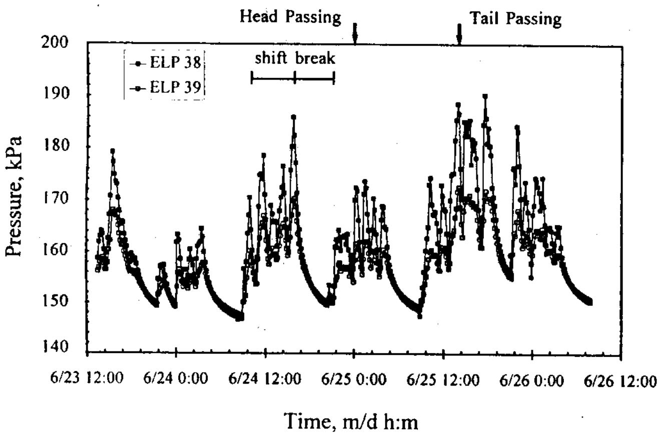


Fig. 21 Piezometer Readings at Section 261T2

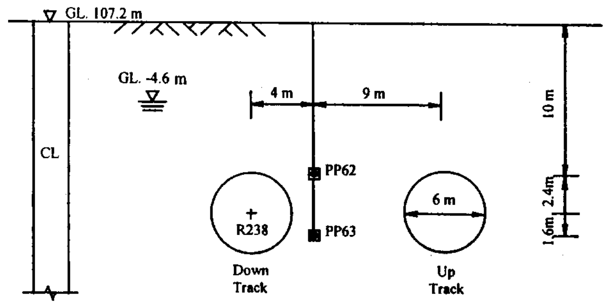


Fig. 23 Soil Profile and Instrument Layout at Section 257T1

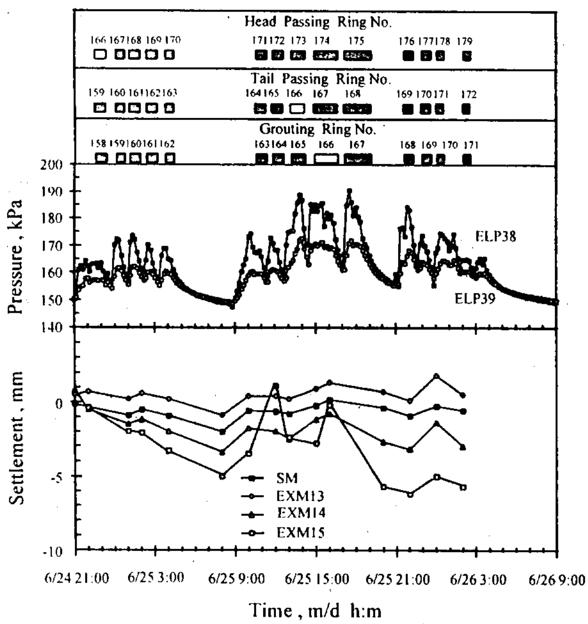


Fig. 22 Pore pressure Response to Tunnelling at Section 261T2

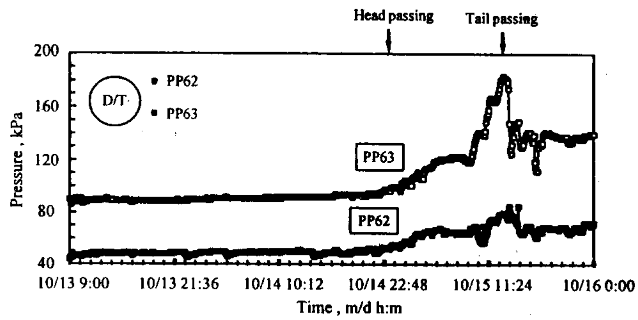


Fig. 24 Piezometer Readings at Section 257T1

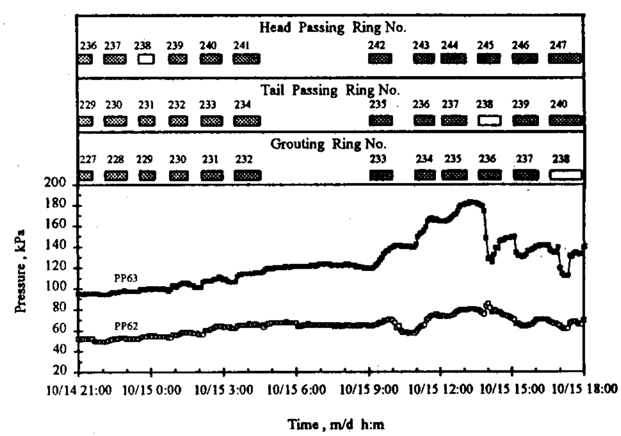


Fig. 25 Pore pressure Response to Tunnelling at Section 257T1

Figure 23 shows the instrument layout and soil profile at Section T1 of CN257 of the Nankang Line and Fig. 24 shows the pore pressure readings obtained (Moh, Hulme and Hwang, 1995; Hwang, Moh and Chen, 1996). The readings are correlated with tunnelling activities in Fig. 25. Unlike the above case for sands, pore pressure responses have two distinct modes: (a) before the passing of the tail, pressures accumulated as each ring was grouted and (b) after the passing of the tail, pressures dissipated with time. During the passing of the tail, there was a clear reduction in pore pressure, presumably, in response to the creation of tail void.

Piezometers installed within 1m from the edge of tunnel are likely to be damaged (Moh, Hulme and Hwang, 1995). It is suspected that the damages were not a result of excessive pressure, rather, they might be due to the fact that the tips were wrapped by grout. For the same reason, piezometers installed on segments would not be able to provide meaning readings and should be avoided.

5.4 Lateral Ground Movements

Lateral ground movements induced by tunnelling are seldom monitored except at placed where the safety of adjacent buildings is of concern or at places where tunnels are close to each other and interaction effects are of interest. In the former case, it is obvious that the worry is that lateral ground movement may lead to tearing of the foundation system or damages to concrete piles which are susceptible to lateral movements. In the latter, refer to Inclinometer SIS14 in Fig. 19, it may be necessary to protect linings of the tunnel which has already been completed.

5.5 Convergences of Linings

It is naturally preferred that convergence of tunnel lining be measured starting as soon as the tail of the shield leaves the segment. However, this is extremely difficult to achieve because of the obstruction by various equipment. For shield tunnelling, measurement can only be made, say, 40m to 50m behind the shield, i.e., a few days later. Therefore, only long-term effects can be measured. The information is useful, for example, for ensuring the safety of a completed tunnel during the driving of subsequent tunnels in vicinity. Excavations or piling carried out in close proximities of tunnels may lead to the relief of geo-stresses and distortion of linings, convergence measurements will be very helpful in quantifying these effects .

Theoretically, the results obtained from convergence measurement can be used to back-calculate the stresses in concrete segments. Even earthpressures acting on these segments can be estimated. However, such studies are rare and inconclusive.

5.6 Stresses in Bracings

When two tunnels are close to each other, the worry is that the driving of the second tunnel may have detrimental effects on the first one which has been completed. This is particularly true for tunnels with concrete linings which are vulnerable to tensile stresses. Bracing is usually required to avoid damages to linings. Bracing is also required for constructing crosspassages between two tunnels. These crosspassages are usually manually excavated. The stresses induced in the bracings can be monitored using strain gauges in a similar way strut loads are monitored in cut-and-cover constructions.

5.7 Pressures in Earth Chambers

For shield tunnelling, pressures in earth chambers are routinely monitored. Usually two to seven total earthpressure cells are available in an earth chamber. Monitoring is automatic and continuous. The information is displayed on a computer monitor in the control room to enable the operator to steer the shield machine properly. It can also be used to correlate ground settlements with chamber pressures. It has been suggested in many studies to maintain chamber pressures at a level corresponding to an earthpressure coefficient of 0.5. Higher pressures were not recommended because of the worry of causing much long-term consolidation settlements. However, recent studies indicate that, as discussed in Section 5.3, the majority of excess pore pressures induced during tunnelling are a result of grouting, not shoving, and it may be beneficial to have higher chamber pressures for reducing immediate ground settlements.

6. BUILDING PROTECTION

Building protection is an essential part of construction programs. To start with, designer estimates zones of influences based on some simple guidances such as the ones shown in Figs. 26 and 27 for cut-and-cover constructions and tunnelling, respectively. The conditions of buildings within the zones of influences are inspected and documented for the purpose of clearing contractor's liability on existing defects. The information can also be used by the designer to establish criteria for the contractors to follow. Potential ground movements are estimated and for buildings which are likely to be damaged, precautionary measures will be taken to reduce the impacts of the works. Notwithstanding all these precautions, instrumentation is required for monitoring the actual responses of buildings. The quantity of instruments installed for protecting adjacent buildings may even exceed the quantity of instruments installed for maintaining the safety of the works.

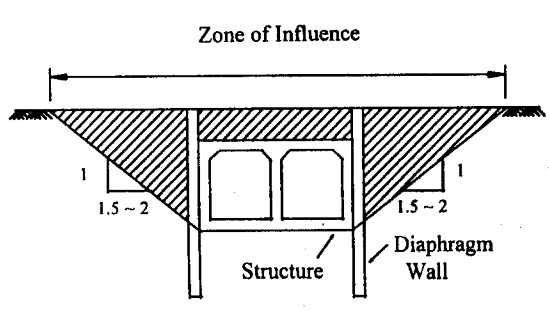


Fig. 26 Zone of Influence for Cut and Cover Constructions, TRTS Project

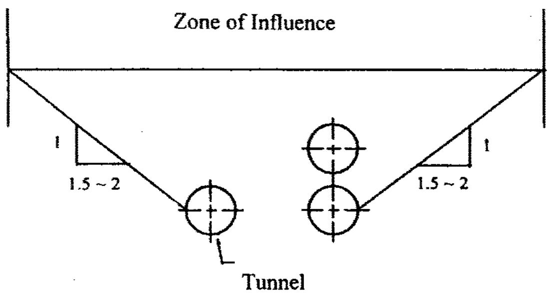


Fig. 27 Zone of Influence for Tunnelling, TRTS Project

6.1 Building Settlements

Settlements of all the buildings within the zone of influence should be monitored. It has been observed that installation of diaphragm wall may cause significant ground settlements. Figure 28 shows a case in which the settlements at the corner of a building induced as a result of installation of diaphragm wall for a subway station and the entrance accumulated to 30mm. The influence of each panel was found to stretch to a distance of 10m and there were a total of nine panels within this distance from the corner in the case shown. It is therefore suggested that monitoring start before the installation of diaphragm walls.

In a few incidents, digging trenches, usually 2m in depth, in very poor ground, or in poor backfill, for constructing guidewalls led to settlements of 30mm or so. Once a trench is dug, guidewall should be constructed and diaphragm wall installed as soon as possible. It is not a good idea to leave a trench open for too long, not even a few days.

It is a common practice to install settlement points on all the columns at the frontage of a building facing the construction and on selected columns at the back. These settlement points shall be installed as early as possible so background disturbances can be determined. Sometimes, a building may undergo settlements even without construction activities. Lowering of groundwater table is known to cause much settlements in many cities. A bad practice is to pump underground water for various construction activities. In one case history, pumping test for determining hydraulic characteristics of the ground during site investigation caused ground in a large area to settle by a few hundred millimeters.

6.2 Tilts

The safety of a building is not governed by settlement alone. In fact, tilting is sometimes a more direct indication of problems. Buildings can tolerate much uniform settlements without causing structural damages. On the other hand, even mild tilt may affect the functioning of elevators and sensitive equipments. Tilt of floors may also affect floor drainage.

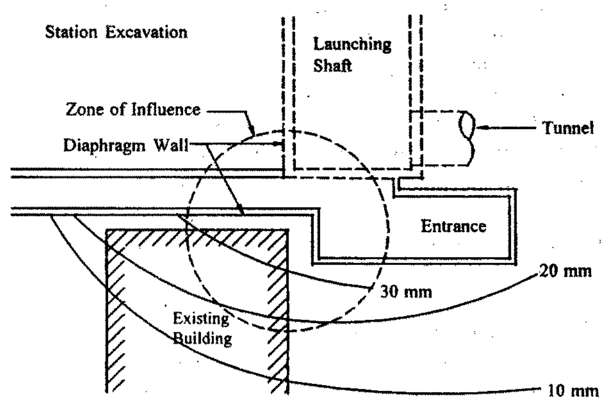


Fig. 28 Settlements due to Diaphragm Wall Installation

The tilt of a building can be monitored by using levels mounted on walls. The tilt of a column can be monitored by plumbing. Electrical type tiltmeters can also be used if precise readings are desired.

6.3 Lateral Strains

Buildings may also be damaged by lateral ground movements. When signs of failures are developing, lateral ground movements can be monitored by inclinometers or, more conveniently, by measuring changes in distance between two points.

6.4 Cracks on Structures

Cracks are usually measured by crack gages. The simplest crack gages consist of two glass plates, one with hairline and the other with a graduated scale, mounted on each side of the discontinuity. Such crack gages are quite suitable for cracks of an order of a couple of millimeters. Narrower cracks are inconsequential and are of little concern. Electrical crack gages are sometimes used when a critical operation is being carried out and accurate/continuous readings are needed.

7. RECOMMENDATIONS

7.1 Type of Instrument

Because of their suitability for automation, electrical instruments are the most preferred type of instruments whenever applicable. Electrical instruments have the most varieties in the family: resistance type, vibrating wire type, linear variable differential transformer, linear potentiometer, variable reluctance, etc. (Dunnicliff and Green, 1988)

Resistance type instruments are unsuitable for underground works for the reason that groundwater may get into the instruments or splices between cables and affect readings. Joint boxes are supposed to be used for splices between cables and infilled with silicone glue to ensure perfect waterproofing, but this procedure is seldom followed. Usually, contractors simply use adhesive tapes to wrap up splices. This may result in malfunctions of instruments. Cables are often totally cut by construction machines. Rejoining of cables will affect readings of resistance type instruments. Cables are also often damaged by sparkles from welding resulting in exposure of electrical wires to groundwater.

The above problems will be less serious with vibrating-wire type instruments which have become more and more popular. In most of cases, vibrating-wire type instruments are even cheaper than resistance type instruments. It is also possible to check whether a surface mounted vibrating-wire type strain gauge is functioning by listening to the twang. Improper welding may damage the gauges. One way to solve the problem is to weld the mounting blocks first with a dummy gauge as a spacer. The real gauge can then be screwed onto the mounting blocks.

7.2 Protection of Instruments

Whatever type instrument is used, the cables should have extra lengths and should be laid loosely to avoid breakage due to tension. Cable splices should be reserved for repair work and should not be planned into a system unless there is no alternative. Wires in a cable should be color-coded, or identified by colored tapes, to make the job easier in case splicing is needed.

Instruments should be clearly marked at the site so they can easily be located and should be well protected to avoid damaged by machines and vehicles. Instruments should also be protected against environment effects. For example, direct sun light will make readings unreliable. It has happened in one case that a building was claimed to "rock" periodically. Pumping of groundwater was first thought to be the reason. It was later found that the tiltmeter was installed on the roof of the podium and the expansion and contraction of water proofing and insulation pads were responsible for the "rocking". The tiltmeter was relocated to the basement and the building no longer "rocked".

7.3 Calibration of Instruments

As a quality assurance procedure, 10% to 20% of sensors shall be calibrated by accredited laboratories before installation. Readout units also have to be calibrated periodically. This often leads to problems. First of all, there are insufficient accredited laboratories available to meet the demands if there are several large projects are ongoing at the same time. Secondly, it is difficult to set performance criteria for different types of instruments. Most of instruments are not included in national or international testing standards. It is therefore suggested for the owner of large project to establish his own laboratory.

7.4 Precautions

It is dangerous to sink holes when excavation is already deep. Drilling from the bottom of excavation at a depth of 34m below surface for installing a grouting curtain was responsible for the flooding of a huge shaft, 28m in diameter, constructed for stormwater sewer system (Construction Today, 1990). If blow-in is a concern, it will be a good idea to specify at least twice as many piezometers as necessary. Hopefully a half of them will survive and it will not be necessary to replace damaged ones. Similarly, if wellpoints are required for lowering the piezometric heads inside a pit, twice as many as needed should be specified. Drilling holes at the bottom of excavation should be carried out only under tight supervision and contingency measures should be available in case of emergency.

7.5 Automation

With the ever-rising labor cost and rapid decline of hardware cost, automation is an obvious and inevitable trend. Automation also helps improving the safety of works as readings can be taken at very short time intervals. This will provide earlier warnings and reduce the chance of accidents. The most notable successful application of automation is probably illustrated in the collapse of NATM tunnels of Heathrow Express near Heathrow Airport, London. Ground monitoring equipment had measured movements "off the scale", alerting tunnellers working in the down-platform tunnel of the impending disaster at about 1am Friday, 21 October, 1994. Twenty minutes later the roof of the new station complex caved in some 20m below (Oliver, et al., 1994). Twenty-five people raced for safety. Immediately afterwards, a 40m stretch of the central concourse excavation caved in followed by a total collapse of the 40m long north running tunnel. The £700,000 electronic instrument around the Heathrow complex was well paid off.

Some of the contractors in the TRTS Project have attempted automation, however, the scale is limited for the reasons that (a) the construction environment is particularly harsh to electronic equipment, such as data loggers and computers, (b) very long cables are required to transmit signals to the office and these cables are difficult to protect, (c) inability of transmitting signals to a far distance. It is believed that these problems will be solved with time. It has been reported that devices are now available to transmit signals through wireless telephones. However, the authors do not have personal experience with such devices.

ACKNOWLEDGMENTS

The authors are grateful to the Department of Rapid Transit Systems of the Taipei Municipal Government for granting the permission of publishing the data presented herein. They also wish to express their sincere thanks to Prof. Yao-Chung Chen of National Taiwan Institute of Technology, Pan Asia Corporation & Chizaki Kogyo Co. Joint Venture, Continental Engineering Corporation and Tekken Construction Corporation Joint Venture, for providing detailed information and to Mr. C. H. Wang of Moh and Associates for stimulative discussions.

REFERENCES

Construction Today (1990). Water pressures stall Tokyo shaft. November

Dunnicliff, J. and Green G.E. (1988) Geotechnical instrumentation for monitoring field performance, John Wiley & Sons, New York, USA

Fellenius, B.H. (1980). The analysis of results from routine pile load tests, Ground Engineering, v.13, No.6, Sept., pp.19-31

Fellenius, B.H. (1984). Ignorance is bliss - and that is why we sleep so well, Geotechnical News, v.2, No. 4, Dec., pp.14-15

Gould, J.P. and Dunnicliff, J. (1971). Accuracy of field deformation measurements, Proc., 4th Panamerican Conf. on SMFE, San Juan, ASCE, New York, v.1, pp.313-366

Moh, Z.C., Hulme, T.W. and Hwang, R.N. (1995). Soft ground tunnelling for Singapore and Taipei MRT systems, Proc., Bengt B. Broms Geotechnical Sym., Singapore, December

Moh, Z.C., Ju, D.H. and Hwang, R.N. (1996). Ground movements around tunnels in soft ground, Proc., Sym. on Geotechnical Aspects of Underground Construction in Soft Ground, April 15-17, London, UK

Hwang, R.N., Liao, C.C. and Fan, C.B. (1996). Lateral earth pressures during deep excavation, Sino-Geotechnics, v.53, February, Taipei

Hwang, R.N., Moh, Z.C. and Chen, M, (1996). Pore pressure induced in soft ground due to tunnelling, Proc., Sym. on Geotechnical Aspects of Underground Construction in Soft Ground, April 15-17, London, UK

Oliver, A., Bishop, P., Winney, M., Parker, D., Doyle, N. and Wheeler, P. (1994). All UK NATM work suspended as Heathrow collapse is investigated, Ground Engineering, v27, no.9, November, 1994, London, United Kingdom

Peck R.B. (1969) Advantages and Limitations of the observational method in applied soil mechanics, Geotechnique, v19, no.2, pp.171-187, Reprinted in Judgment in Geotechnical Engineering: The Professional Legacy of Ralph B. Peck (1984), J. Dunnicliff and D. U. Deere (Eds.) Wiley, New York, pp.122-127

Peck, R.B. (1969a). Deep excavations and tunnelling in soft ground, Proc., 7th ICSMFE, State-of-the-art volume Mexico City, Mexico, pp.225-290

Saxena, S.K. (1974). Measurement performance of a rigid concrete wall at the World Trade Center, Proc., Int. Conf. on Diaphragm Walls and Anchorages, Inst. of Civil Engineers, London, pp. 107-112

Soares, M.M. (1983). The instrumentation of a diaphragm wall for the excavation for the Rio de Janeiro Underground, Proc., Int. Sym. on Field Measurements in Geomechanics, Zurich, K. Kovari (Ed.), Baıkema, Rotterdam, v.1, pp553-563

Wolosick, J.R. and Feldman, A.I. (1987). Reliability of bending moments backcalculated from inclinometer measurements, Geotechnical News, v.5, No.3, September