HKIE Seminar on Geotechnical Aspects of Mass Transportation Systems

Opening Keynote Address by
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Director of Highways, HKSAR Government

Mr Chairman, Ladies and Gentlemen

Let me begin by thanking the Geotechnical Division of the Hong Kong Institution of Engineers for inviting me to deliver a keynote address in this Annual Seminar. Today I shall share with you some geotechnical challenges in our major mass transportation systems, including highway and railway projects, over the past years and in the decades to come.

The usage of Hong Kong's roads is among the highest in the world. No doubt, this heavy road usage is a reflection of the prosperity of Hong Kong. To cope with our rapidly increasing population and to sustain our growth and development, we have undertaken a number of major transportation projects since the 1970s. The completion of the first Cross Harbour Tunnel nearly 30 years ago and the subsequent opening of the first Mass Transit Railway (MTR) line to Kwun Tong dramatically changed the travelling pattern of many Hong Kong people. The transport network was further improved significantly in the 1980s with the completion of many major projects such as the Hong Kong Island Eastern Corridor, the Eastern Harbour Crossing, the New Territories Circular Road together with the MTR Tsuen Wan and Island Lines.

With the building of the new airport in the 1990s, the previously insolated Lantau Island is now connected to the urban areas and the New Territories by a massive network of highways and railways. In fact, many of the projects in the Airport Core Programme (ACP) are for the purpose of mass transportation, for example, the North Lantau Expressway, the Lantau Link, the West Kowloon Expressway, the Western Harbour Crossing, the Route 3 Highway and the Airport Railway. Other projects, such as the West Kowloon Reclamation and the Central Reclamation, provided land for some of the transport facilities (Figure 1).

On completion of the ACP, we have continued to expand and improve our transport infrastructure to meet the ever increasing traffic demands, particularly in cross-boundary traffic. Major highway projects already completed include the Ting Kau Bridge and the Tsing Yi North Coastal Road.

In view of the shortage of land in Hong Kong, these transport projects have been built on difficult terrain. For example, Tuen Mun Road was constructed on steep hillside involving the formation of major cut and fill slopes. In some areas, large scale reclamation has to be undertaken to provide land for various projects, including transport infrastructure. The viaducts of the Yau Ma Tei Interchange built on the West Kowloon Reclamation were good illustrations. Many other roads were constructed on elevated viaduct structures, for example, the interchange at Route 3 linking the Ting Kau Bridge with Tai Lam Tunnel. Sometimes slip roads were constructed on embankments, as in the case of the West Kowloon Viaduct at Mei Foo Interchange. Other roads were constructed in tunnels, such as the Route 3 (Country Park Section). A number of long span bridges and underwater tunnels had also been built over or under coastal waters and sea channels.

The planning, design and construction of these transportation systems have presented major challenges to the geotechnical profession. This is particularly important in Hong Kong, where large tracts of terrain have been weathered to substantial depths leaving thick soil units on steep slopes. The weathered soil profiles are highly variable in terms of depths, composition and material properties. The heavy annual rainfall has created significant hazard from landsliding and rockfall activities affecting traffic corridors. The extensive slope stabilization along Tuen Mun Road well demonstrated this point.

In addition to slope-related hazards, other geotechnical problems to be resolved during design and construction of transport infrastructure projects are related to the stability of foundations. Examples are the huge foundation and anchor structures of the Tsing Ma Bridge and the bored pile foundations under busy road for the Wong Chuk Hang Flyover.

For tunnels and underground structures in urban areas, geotechnical engineers are engaged in the estimation, monitoring and control of settlement of adjacent ground and buildings during excavation and dewatering. The deep excavation in Tsuen Wan for the West Rail adjacent to tall residential buildings and busy road viaducts required special attention. For new reclamations on which highways, railways and airport are constructed, geotechnical consideration would have to be given to long term ground settlement, particularly where soft marine deposits and other problematic soils are present in the seabed. Sometimes preloading and compaction of the newly reclaimed land are necessary to reduce future settlement.

Geotechnical input is also required in the solution of erosion, flooding and ground contamination problems. Examples are the erosion of the natural terrain above the North Lantau Expressway at Tung Chung, the flooding of low-lying areas in the Northern New Territories after torrential rain and the excavation of contaminated soil under the old Kai Tak Airport for the South East Kowloon Development.

As you may have noticed in the previous examples, geotechnical engineering has played a significant role in our mass transportation projects. The highly variable ground conditions of our local terrain can never be completely known before construction. Our geotechnical engineers have to make reliable predictions, monitor the construction and, where problems occur, decide on the corrective actions to be taken, usually within tight time scale. The successful completion of all these projects has borne witness to the high degree of skill and dedicated efforts of our geotechnical profession.

Another point I would like to mention is the rising public expectation on the environmental standards of our public works projects and this includes visual appearance of our slopes. The Government has now committed to landscape every new slope formed. We are also trying out new techniques to green our existing roadside slopes. An example is the recent application of a proprietary system imported from Japan for greening a major cut slope along the Tsing Yi North Coastal Road. I am certain that our efforts in slope greening will improve the image of our transport network and make Hong Kong a more pleasant place to live in and to visit.

Now I like to turn to future road projects. The Government has already committed to maintain Hong Kong's position as Asia's World City. We must equip it with a world class transportation system. In the coming 10 years, Government is planning to invest more than $100 billion on road projects (Figure 2). Our agenda includes the Shenzhen Western Corridor and Deep Bay Link, Route 9 linking Tsing Yi and Shatin, and Route 10 linking North Lantau with Western New Territories. Other projects include the Central Kowloon Route and Central Wanchai Bypass. These major highway projects are scheduled for completion between 2005 and 2012.

With regard to rail projects, the Government together with the two Railway Corporations will invest $200 billion over the next 15 years on a massive expansion of the railway network (Figure 3). For the first phase, six new rails will come into place in the next five years at a total cost of about $100 billion. Starting from later this year, we will see the birth of one new rail nearly every year. They are the MTR Tseung Kwan O Extension opening in August 2002, West Rail in 2003, Ma On Shan and Tsim Sha Tsui Extension Rails in 2004, Penny's Bay Rail Link in 2005 and Sheung Shui to Lok Ma Chau Spur Line in 2007.

For the next phase of railway development up to 2016, there is plan to invest another $100 billion on six additional rail projects to relieve bottlenecks in the existing railway systems, to provide rail service to new development areas and to meet cross boundary passenger/freight demands. These projects include the Island Line Extensions, Shatin to Central Link, Kowloon Southern Link, Northern Link, Regional Express Line and Port Rail Line. When complete, the total length of our present rail network system in the Territory will be almost doubled.

This total investment of nearly $300 billion on road and rail projects in the mass transportation system will mean numerous opportunities for the geotechnical engineering profession. I am confident that, with the wealth of experience you have gained in the past, you will continue to make outstanding contribution to the realization of our future transport programme and the prosperity of Hong Kong.

As for today, I am sure you will have much to gain from the presentations and discussion of the intense geotechnical efforts that went into mass transportation systems. I wish you every success in the Seminar.

Thank you again for inviting me to make this keynote address.

Airport Core Programme (ACP) projects

1. **Chek Lap Kok Airport**
2. Tung Chung Development
3. **North Lantau Expressway**
4. **Lantau Link**
5. **Route 3**
6. West Kowloon Reclamation
7. **West Kowloon Expressway**
8. **Western Harbour Crossing**
9. Central Reclamation
10. **Airport Railway**

Note : Projects for mass transportation are highlighted

Figure 1 Projects in the Airport Core Programme (ACP)

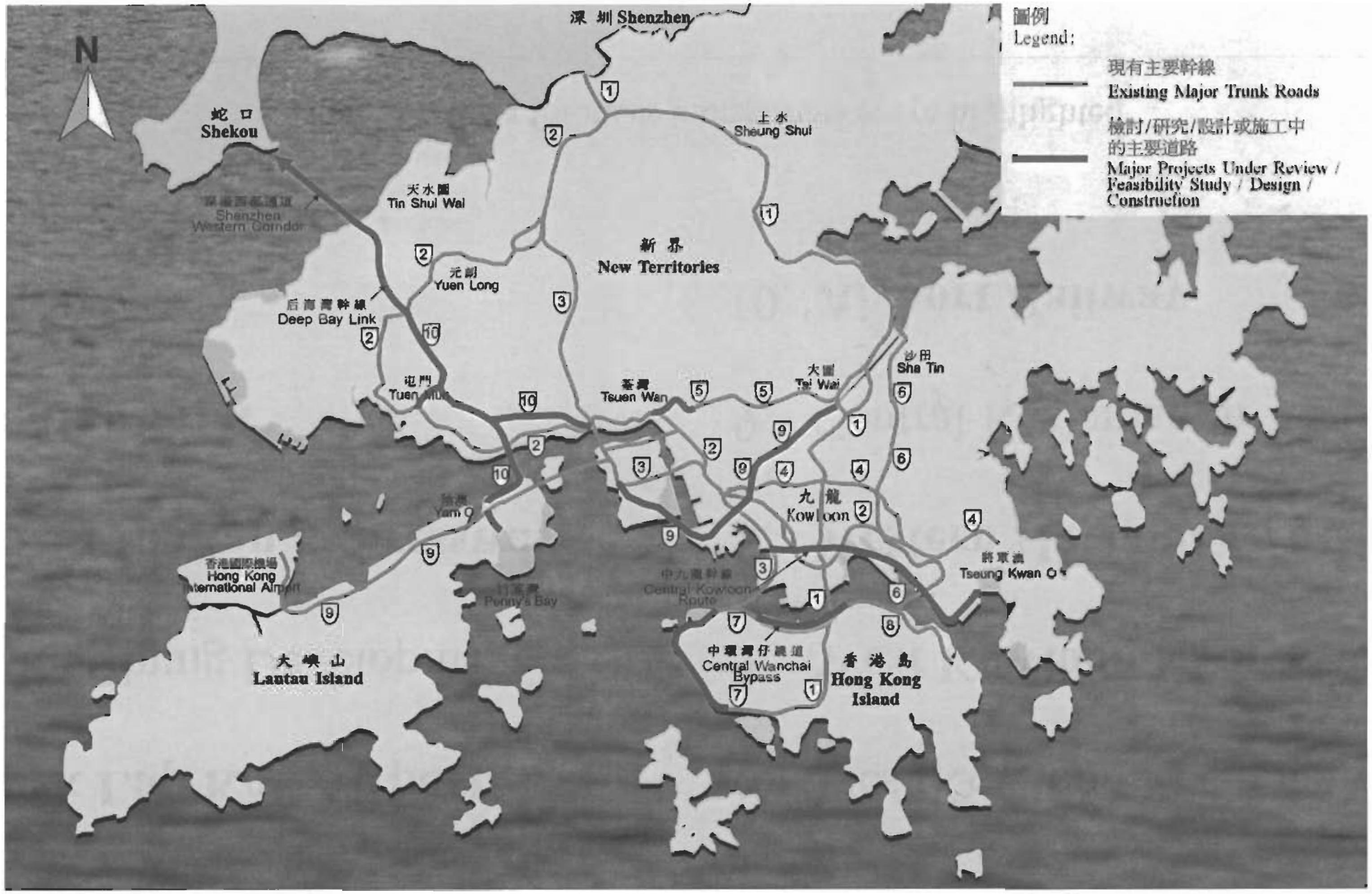

Figure 2 Forthcoming Major Road Projects

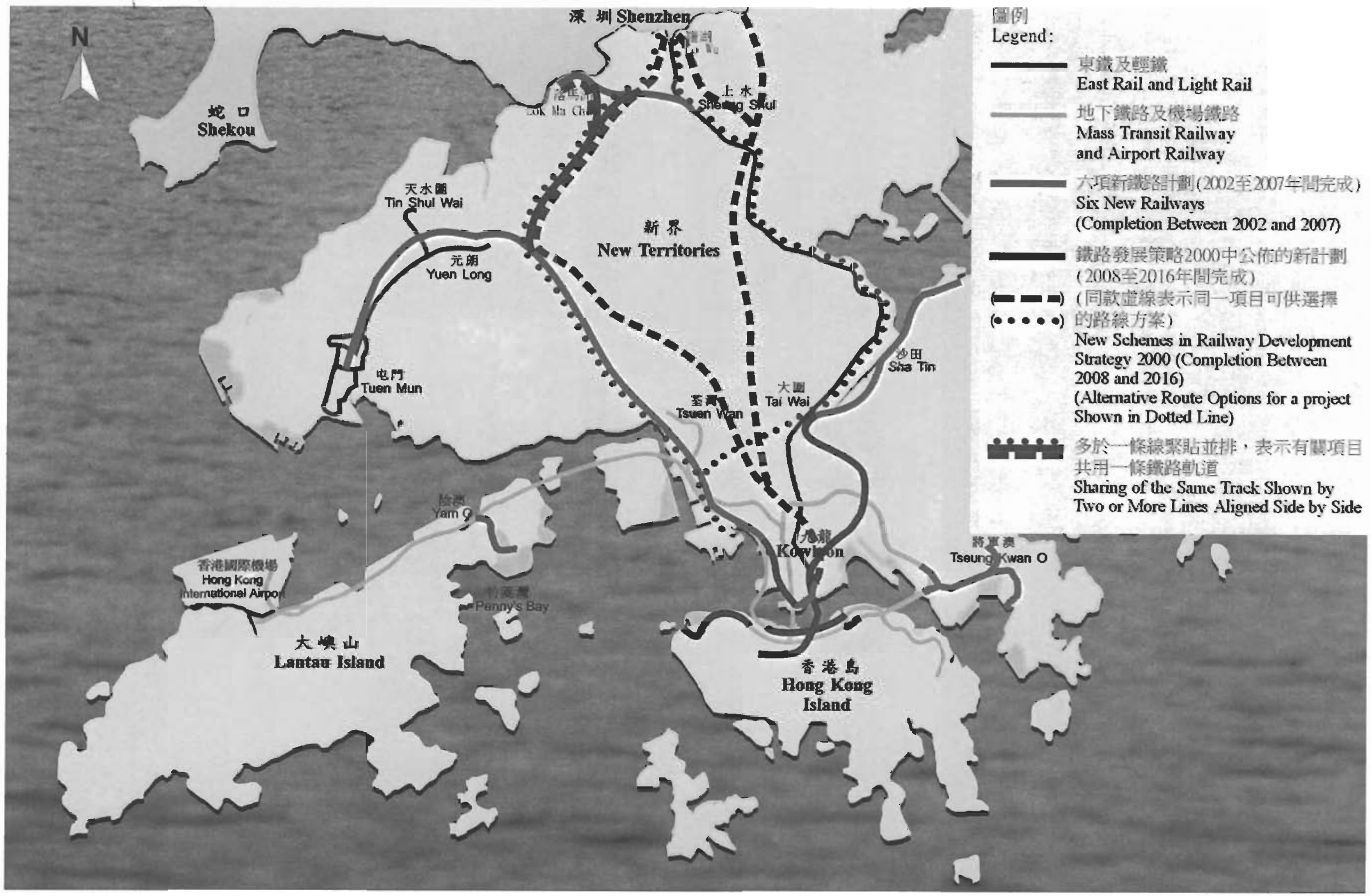

Figure 3 Forthcoming Railway Projects

NUMERICAL MODELING IN GROUND ENGINEERING FOR TRANSIT PROJECTS

L.J. Endicott[1]

ABSTRACT: During the last thirty years the computer has developed from a research tool at universities to a standard piece of office equipment. Thirty years ago, one had to queue up for time on a machine, nowadays most of us turn on the computer in the office each morning before sitting down. Numerical methods have been transformed. Graphical solutions for slope stability are of the past along with slide rules and many of the closed form solutions, and complex situations can be modeled repeatedly for a multiplicity of scenarios.

INTRODUCTION

This paper traces the development of the application of numerical modeling to ground engineering for transit from the research applications of a generation ago to some of the latest projects currently under way in Hong Kong.

COMPUTERS AS A RESEARCH TOOL

Electronic computers, as we know them, were developed secretly in order to crack military codes generated by mechanical encrypters. After the war, over fifty years ago, computers were given to universities and were adopted as research tools. In the 1960s, few consultants used them. One of the earliest uses was in about 1968 when Maunsell set up Glen Computing in order to process survey data, mostly for roads, and had spare capacity to hire out to other consultants. They opened a computer bureau in Victoria Street, the favoured address for the consultants of the time.

In the universities, computers were being used. For example, at Cambridge University, under the direction of Prof. Ken Roscoe, x-rays or photographs were taken of plane section models which traced movements within the plane section throughout the tests. A PDP8 computer, with an 8k processor and coded in binary, was used to count optical fringes on a viewer and measure coordinates on the photographic plates to within one micron, Ref. Endicott (1971).

I used this equipment to determine displacements within a model slope during failure. At the time, there were virtually no solutions for stress distributions within slopes with which to compare my test results. Slope stability was being analysed as solid body equilibrum on drawing boards by graphical methods. Dr. Brian Simpson was writing his finite element program at the time so I had to analyse the data by other methods. I borrowed from continuum mechanics, firstly plastic flow rules, Ref. Prager (1959), and secondly solution by the method of characteristics, Ref. Sokolovski (1965). I was able to use an IBM 1130 with a 28k cpu and paper tape data input.

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SOIL STRUCTURE INTERACTION

In 1975, tenders were invited for the modified initial system, Admiralty to Kowloon Bay, of the Hong Kong Mass Transit Railway. Governed by the Contractor's desire to minimize costs, soil structure interaction was adopted for the design of underground structures. Since a seven function pocket calculator cost two month's salary for a designer at the time, calculations were performed with a slide rule, Ref. Benjamin, Endicott and Blake (1978).

The successful award of four of the contracts, including Choi Hung and Diamond Hill Stations, to Paul Y Construction required detailed design calculations to be carried out quickly and repeatedly. Diaphragm walls were a large proportion of the costs and, for reasons of economy, the detailed design had to include interaction between diaphragm walls and the adjoining soil. A beam on springs model with strain softening and plastic yield for the soil reaction on both the active and passive sides was developed in-house in 1978. It was called DIANA and was a standard design tool until the early 1990s. Figure 1 shows the principles of the beam on springs model.

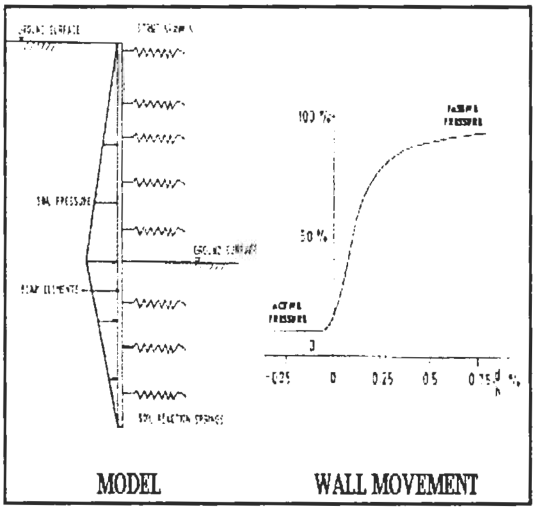


Figure 1: Beam on Springs Model

At this time, ground engineering lagged behind structural engineering with respect to use of numerical methods. Structural analysis for large buildings and bridges were adopting large space frames and shell elements in computer programs.

GROUND AS A CONTINUUM

In the 1980s personal computers became available and software was being produced by software companies. Programs for slope stability, for seepage analysis, and for consolidation came onto the market.

Several programs were available which treated the ground as a continuum, either by finite elements or by finite differences, but material properties were limited to elastic behaviour and the Mohr-Coulomb yield criterion.

The first use of a continuum analysis for private building works approved by the Building Authority in Hong Kong was for the development of No.1 May Road and the associated road widening in 1986, Ref. Endicott and Cheung (1991). The finite difference program, FLAC, Ref. Cundall (1987), was used and computation was carried out on a 286 processor. The grid used in the analysis is shown in Figure 2.

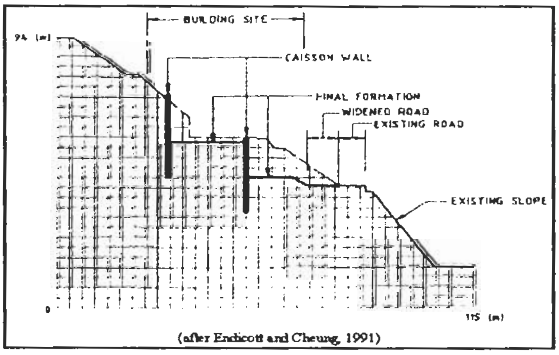


Figure 2: Hillside Site

The reason for using numerical analysis was the complexity of the interaction between the ground and two walls in cascade, the one above the other such that the passive pressure in front of the top wall was applied to the back of the lower one. Since the walls were reasonably long and straight, a 2-D model was quite appropriate for this project.

This project is probably another first. It is probably the first project for which numerical modeling was used for the design which was built and has since been constructed, it has since been demolished and has been rebuilt to another design with a higher plot ratio.

Applications of FLAC in 2-D for tunneling were not so straightforward. Analysis of a failed railway tunnel in Singapore in 1988 during construction, Ref. So and Endicott, (1990), was facilitated by the ability of the program to model plastic yield and to accommodate large displacements. Figure 3 shows a plot of the displacements computed for the tunnel. However modeling a longitudinal section required a transformed section to avoid modeling the tunnel void as a semi infinite slot.

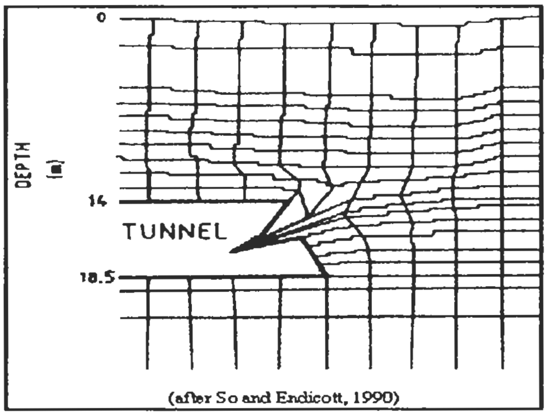


Figure 3: Analysis of Tunnel Failure

In this case, the program alone in 2-D would not have been appropriate without careful determination of properties of transformed sections in order to model out of plane effects. It was considered that the shallow depth of the tunnel and proximity to the rock would not have lent itself to the use of an axis-symmetric model.

Monitoring of the performance of the construction at No. 1 May Road and comparison with the design predicted movements gave confidence in the use of numerical modeling. In 1988 to 1990 there was the opportunity to use FLAC to model the diaphragm walls for some 20 of the underground stations and a similar number of tunnels for the Taipei Rapid Transit System.

One of the biggest problems at the time was with checkers who expected designs to be based on Terzaghi and Peck's trapezoidal envelopes of soil loading for braced excavations, Ref. Geoguide 1, (1982). Fortunately the successful use of the computer program for sites in Hong Kong and Singapore met with the checker's approval.

Another problem at this time, was that people were convinced that wall friction would be less than phi, the angle of friction of the ground. Therefore numerical modeling had to include slip elements on the face of the wall with a reduced value of 2/3 phi. Yet, FLAC solutions show a plane of maximum shear stress on the wall, the principal stresses are at 45 deg to the wall and the ratio of shear stress to normal stress is arc tan sine phi. That is, it is about 2/3 phi. By introducing a slip element with soil having 2/3 phi results in an output with 4/9 phi, the 'slip' effect was double counted, Ref. Endicott and Cheung (1990).

The lesson in this case was "look at the results".

TUNNEL CROSSOVERS

For over a decade, numerical methods for modeling in plane strain were used for many transit projects. Meanwhile, transit projects in metropolitan areas became progressively more complicated and it became apparent in the early 1990s that 2-D modeling was being stretched to the limit for want of the availability of a 3-D code, which was suitable for commercial use, and hardware of sufficient capacity to run it. The preliminary design for the Singapore Underground Roads System was a case in point. The project involved a circular underground road system which had to cross the existing MRT underground rail system at a series of locations.

Where the proposed new road tunnels had to cross over the existing MRT bored tunnels there were two cross sections, one for the SURS tunnel and another for the MRT. Again, FLAC 2-D was used but both cross sections had to be considered interactively. Figure 4 shows the two sections diagrammatically.

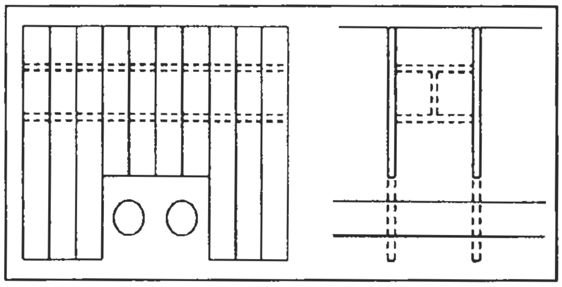


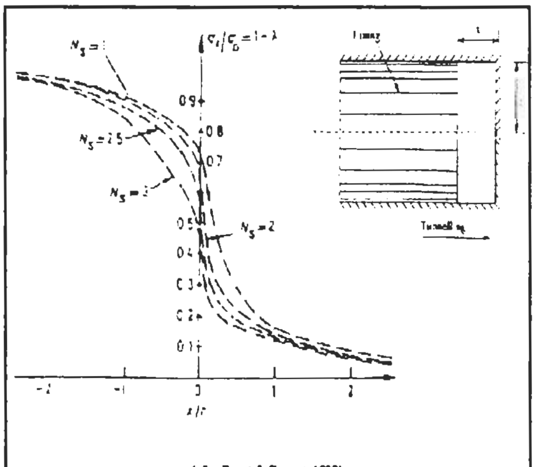
Figure 4: Schematic Section for Crossing

Cross interaction was modeled by taking two complementary cross sections. The one in the plane of the existing bored tunnels was used to develop an equivalent soil and structure stiffness which was put into the model of the other cross section to model out of plane effects. The technique was a little clumsy but the 2-D models were relatively fast to run. Each run took about half an hour on a 386 PC and a range of values could be tested. The modeling showed that displacements of the existing rail tunnels could be kept within the prescribed limits of 20mm. It was anticipated that, by the time the detailed design was to go ahead reliable 3-D codes would be available and computers would be able to run them in reasonable periods of time.

NATM ANALYSIS

NATM is a much heralded method of analysis of tunnel and soil interaction for design and construction which has gone a long way towards economical rock tunneling in a range of types of ground. Ten years ago NATM was being adopted for relatively large spans in soil. The collapse of the Heathrow Tunnel in London Clay brought the techniques of design and construction under scruitiny. Work done on analysis of the collapse, Ref. Fraser and Endicott (unpublished), was typical of the times. 2-D analysis could only approximate 3-D effects. The interaction between the ground and the lining, often erected in a series of stages, was carried out in plane strain section across the tunnel. The changes in stress in the ground prior to the tunnel reaching the plane under consideration were adopted on an empirical basis. Ref. Panet and Guenot (1982), as shown in Figure 5.

It was assumed that of the order of 70% of the overburden was shed before the tunnel arrived. A plane section across the tunnel was modeled for interaction between the lining and the stiff London Clay for only about 30% of the overburden pressure, as shown in Figure 6. Trial calculations were able to match the performance of the tunnel.



(after Panet & Guenot, 1982)

Figure 5: Virtual Support Pressure versus Distance to Face

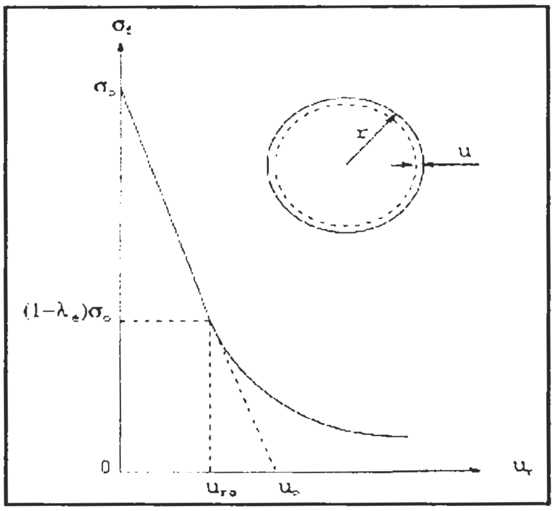


Figure 6: Ground Characteristic Curve

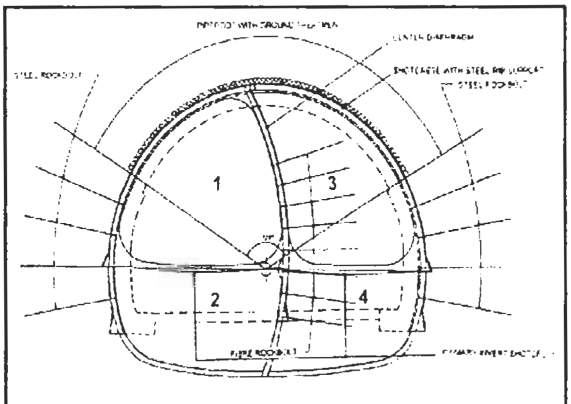
The analytical work made conclusions about the collapse. However, the derivation of stress relief before the face was empirical and although there were a number of papers on tunnel convergence at and after the face, calibration from one any site was not necessarily transferable to any other site condition.

NATM IN HONG KONG

An example of the application of NATM in Hong Kong was for the design and construction of the Tai Lam Road Tunnels, Ref. Endicott, et al (2000). Two three lane road tunnels, of 16.5 m span, and a ventilation tunnel, of 13 m span, were excavated through CWG under a catchwater and, needless to say, in a hurry.

It was an obvious situation to adopt the NATM approach but there was no prior experience of tunneling using NATM in CWG in Hong Kong from which to get an empirical stress reduction.

Empirically based methods of determining stress reductions were available for rocks and for soils from overseas, but there was uncertainty whether CWG would fit either category. Analysis was carried out for a number of scenarios and careful site monitoring was adopted for control and site verification of design assumptions. Figure 7 shows a tunnel cross section. Some 15 stages of excavation were modeled.



(after Endicott, Ashley and Tattersall, 2000)

Figure 7: Tai Lam Road Tunnel Section

Fortunately the work went ahead very much as planned. The result was a rapid construction of the tunnels, ground movements were within the predicted values, and there were no problems with the important catchwater. The lesson learned from this exercise was that the NATM method can be used successfully in CWG and the method can be suitable for design analysis and control during construction for large span tunnels in Hong Kong provided that care is taken.

Shortly after the completion of the Tai Lam Road Tunnel, the Tai Lam West Rail Tunnel was awarded for design and build in the same part of Hong Kong. For much of the design of the twin track rail tunnel, the methods used for the road tunnels could be adopted. However, it was anticipated that the rail tunnel would intercept the Sham Tseng fault close to a water tunnel. The designer was required to consider a 16 m span excavation passing through weathered faulted granite. Experience of having completed the road tunnel on a nearby alignment was incorporated into the thinking for this project. In the worst scenario, completely weathered material was assumed to be present for as much as thirty metres, there were over 400 metres of overburden and the fault may be charged with ground water under pressure. Conventional design involving soil overburden pressures required multiple headings and very thick linings. The contractors wished to tunnel through in one pass.

The design approach involved several steps. These were to evaluate the range of ground conditions that should be considered and the corresponding engineering properties for each. Next the degree of convergence and overburden stress reduction ahead of the tunnel face was evaluated. The method used was based on empirical methods for rock tunnels, Ref. Hoek and Brown (1980). The estimate of overburden reduction was repeated for the whole range of possible ground conditions and for a corresponding range of temporary face supports. For the cases with poorer ground, fibre dowels were proposed in the face and, in the event, these were adopted for construction. Because the tunnel was relatively deep, it was thought that the initial ground conditions could be considered to be sufficiently uniform that the convergence and stress relief during tunnel advance could also be modeled with an axis-symmetric model.

Figure 8 shows a plot of convergence during tunnel advance using an axis-symmetric model. A third approach was considered. PHASE, 2-D version, was used to model the overburden reduction due to ground convergence ahead of the face by considering a cross section of the tunnel within the fault whereby the stiffness of the ground within the outline of the tunnel was progressively reduced thereby simulating the tunnel advance process out of plane. These three methods provided estimates of the overburden reduction which, with judgement could be adopted for the remaining stage of design.

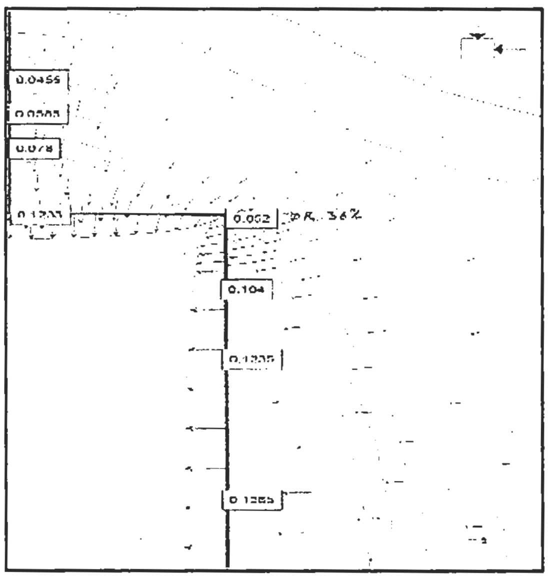


Figure 8: Axis-Symmetric Model

Figure 9 shows a summary of the overburden relaxation estimates which included plastic yielding. The last stage of the design process was to adopt the estimated overburden reduction, which was very high, of the order of 90 to 100%, and to model the interaction between the ground and the support system. This stage was modeled by FLAC 2-D version. Figure 10 shows the estimated total displacements.

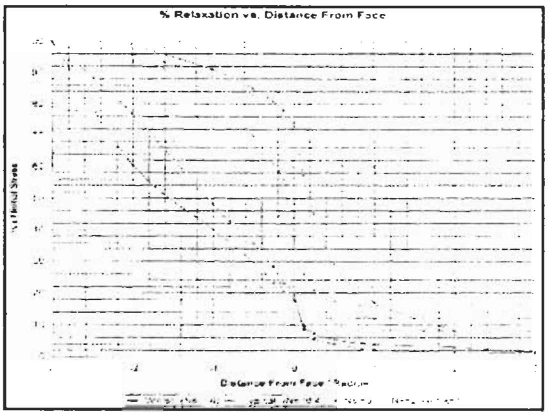


Figure 9: Relaxation of Overburden

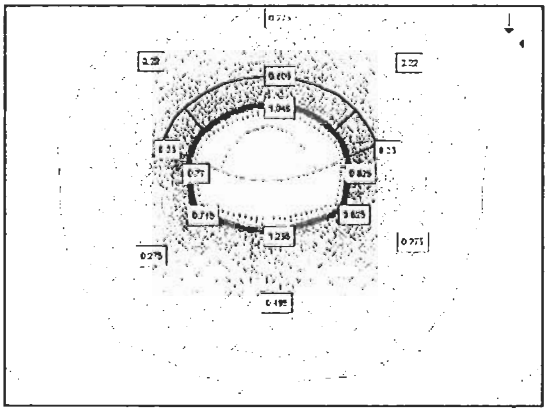


Figure 10: Estimated Ground Movement

During construction it was found that the fault was not as wide or as weathered as expected and the performance of the ground and the support system was within the design expectations. Excavation and support erection was accomplished in one pass albeit at a slow and cautious rate.

The lesson learned from this exercise was that Pentium processors can run quite large models for many alternative cases. More importantly the exercise demonstrates that a computer can not be used simply as a black box whereby ground investigation data is fed in and drawings come out. On the contrary, the design process must remain under the control and within the judgement of the designer. Numerical methods can be adopted successfully, but only for some of the steps in the design process. In some cases alternatives have to be weighed, decisions made, and adopted where appropriate. Overall the designer drives the process and not the machine. The numerical methods are tools and tools only. For this project the most significant effect, the reduction of the overburden by between 90 and 100% ahead of the face, was the most difficult to determine with the tools available. The analysis of the interaction between the temporary support and the remaining 0 to 10% of the overburden pressure remaining in the ground was relatively straightforward by numerical modeling. It was concluded that improvements could be made if the behaviour of the ground ahead of the face could be better understood and modeled with confidence and with better tools.

For a task of this complexity, it is good practice to consider a range of possible scenarios and it is certainly a good practice to run simple models first to understand the mechanisms involved and then develop the more detailed models afterwards.

HARD ROCK TUNNELS

Numerical modeling has been adopted for many applications in hard rock tunnels. Representations of the rock mass range considerably. On the one hand, mass properties are assumed to be dominated by jointing, and programs such as UDEC, have been in general use as design tools for nearly ten years. UDEC provides a discontinuous medium with specified joint orientations with control on spacing and persistence. Figure 11 shows a UDEC analysis of a flat arch design performed in 1998. A plane section is depicted with computer generated joint distributions and corresponding displacements. On the other hand, rock mechanics methods of characterization can be used to determine equivalent required support pressures and these can be applied as applied loadings to a non-interactive structural analysis program for lining design. Figure 12 shows an application of

SAP 2000, from 1998, used to estimate the interaction between a platform tunnel lining and the lining for a perpendicular adit. These methods are now commonly in use.

Figure 11: Flat Arch Model

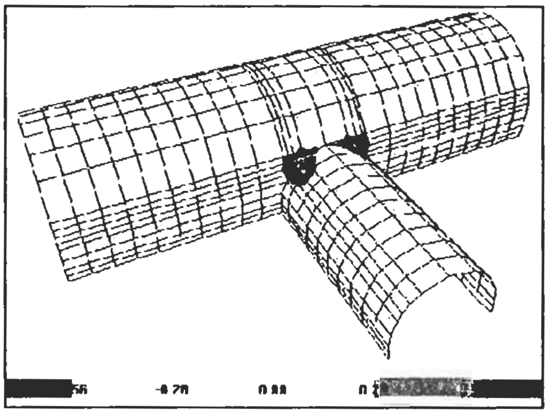


Figure 12: Intersecting Lining Model

3-D APPLICATIONS

Computer programs for modeling the ground in 3-D have been available for a few years in a user-friendly form. A recent example of the use of 3-D modeling for a transit project concerns a proposed excavation over the top of running tunnels in Hong Kong as an extension to Tsim Sha Tsui MTR Station.

Diaphragm walling was proposed with an excavation 10 metres deep to within 1.5 metres above the existing tunnels. Initial estimates using 2-D analysis for the walls and excavation, modeling the whole length of the structure, predicted that the tunnels would rise during excavation well in excess of the allowable movements for MTR structures in Hong Kong. It was decided to explore the use of staged construction, working in short lengths at a time, for which 3-D effects would be very important. FLAC, 3-D version, was used to model the construction of the walls, the excavation and shoring and the construction of the permanent works and backfilling. Ref. Suraj De Silva, Cheung and Cheung (2002).The analysis made use of the understanding of the mass properties of the ground which has been accumulated over the many years of numerical modeling of subway construction in Hong Kong and resulted in predicting tolerable movements. The 3-D grid used in the analysis is shown in Figure 13.

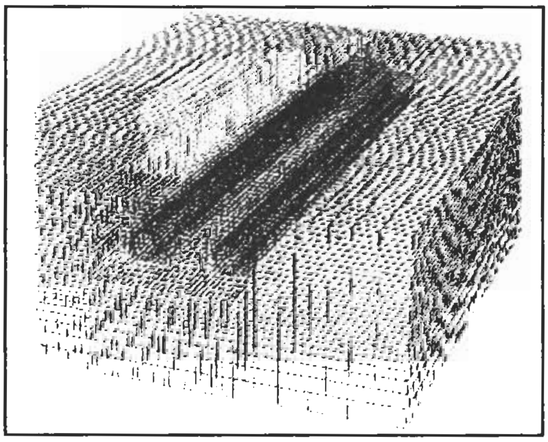


Figure 13: 3-D Analysis of Excavation

This example showed that 3-D effects need to be taken into account for some of the tricky situations where a simple approach can be overly conservative. The ability to run quite complicated models is certainly a lot more con venient than having to adopt 2-D models with out of plane stiffness such as were adopted ten years ago. However The experience of having used 2-D analysis for this type of ground lends confidence to the validity of the 3-D modeling. Unfortunately, the project has not yet been built and the results of the modeling cannot be compared with observations in the yield yet.

CONCLUSIONS

In the limited time available I have only touched on a few examples of applications of numerical analysis for underground transit ground engineering. What is staggering is the tremendous strides in the development of software and hardware such that complicated situations can be modeled relatively conveniently. In the last 27 years of mass transit construction in Hong Kong, designers have progressed from slide rule calculations for plane sections of diaphragm walls, to 3-D numerical modeling with a variety of material properties available for the ground. It has been fortunate that as the mass transit underground construction work in Hong Kong has become progressively more complicated so the numerical methods available to designers, both in terms of user-friendly software and hardware, have grown in parallel. In 1975, when there were few underground structures, simple methods were sufficient. Now, 27 year later, designers and builders are trying to weave new underground structures above, below, and in between other underground structures. In this sector of ground engineering, I can expect that the future will hold further complexity both in the field, as our underground space becomes more developed, and in the numerical methods which appear to still have scope for further development.

REFERENCES

Benjamin, A.L., Endicott, L.J. and Blake, R.J. (1978) "The Design and Construction of some Underground Stations for the Hong Kong Mass Transit Railway System", Proc. Structural Engineering. No.1, Vol. 56a, 11-20.

Cundall, P.A. (1987) "FLAC – Fast Lagrangian Analysis of Continua, User's Manual" Itasca Consulting Group.

Endicott, L.J. (1971) "Centrifugal Testing of Soil Models" Ph.D Thesis, Univeristy of Cambridge.

Endicott, L.J. and Cheung, C.T. (1991). "Temporary Earth Support". Proc. Seminar on Lateral Ground Support, Geo. Div. H.K.I.E., May 1991, 39-49.

Endicott, L.J., Ashley, J.J. and Tattersall, J.W. (2000) "Design of Tunnel Portals and Initial Drives for the Route 3 Tai Lam Highway Tunnels, Hong Kong. Proc. Conf. GeoEng 2000, Nov. 2000, Melbourne.

Frazer, R.A. and Endicott, L.J. (unpublished). Paper in preparation.

Geoguide 1 (1982). "Guide to Retaining Wall Design". G.C.O., Hong Kong Government.

Hoek, E. and Brown, E.T. (1980). "Underground Excavations in Rock". The Institution of Mining and Metallury, London.

Panet, M. and Guenot, A. (1982). "Analysis of Convergence behind the Face of a Tunnel", Proc. Conf. Tunneling 82, The Institution of Mining and Metallurgy, 197-204.

Prager, W. (1959). "An Introduction to Plasticity", Addison-Wesley, 1959.

So, W.K. and Endicott, L.J. (1990). "Analysis of a Failed Tunnel in Soft Ground". Proc. 10th Southeast Asian Geotechnical Conf., Taipei, 16-20th April 1990, Vol. 1, 563-568.

Sokolovski, V.V. (1965). "Statics of Granular Media", Pergamon, 1965.

Suraj De Silva, R.H.G, Cheung, K.W, and Cheung, C.T. (2002). "Design of MTRC Tsim Sha Tsui Station Concourse Extension Works- Geotechnical Aspects", paper submitted to this Seminar.

滲流對擋土結構水土壓力的影響

李廣信　、　劉早雲

Influence of Seepage on Water and Earth Pressures on Retaining Structures

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撮要

鐵路與公路兩側常常修建擋土構造物。土中水不僅對擋土構造物作用水壓力，而且由于土中水的滲流也明顯影響其上的土壓力。本文對于地下水的形態（滯水、潛水及承壓水），施工中排水方式，土中正負超靜孔隙水壓力等情況進行了分析計算。結果表明，在上層滯水情況下，用水土分算計算水土壓力一般可以接受；在有承壓水情況下，滲流作用中的擋土牆被動土壓力會大大減少；在施工時支援結構的外側排水相對于內側排水，更能利用滲流力提高被動土壓力，降低主動土壓力，提高結構的穩定性；土中正的超靜孔壓增加擋土結構上的水土壓力，負超靜孔壓減小水土壓力。計算分和析還表明，除了一維滲流的大多數情況下，水土壓力宜于用庫倫土壓力理論計算，而不宜于用朗肯土壓力理論計算。

INFLUENCE OF SEEPAGE ON WATER AND EARTH PRESSURES ON RETAINING STRUCTURES

Li Guangxin[1] & Liu Zaoyun[2]

ABSTRACT: In retaining structures of foundation pit, water and earth pressures are generally dominated by seepage of water in soils. In this paper, the seepage influence is calculated and analyzed under various conditions by using Rankine's theory and the graphic method of Coulomb's method, and it is found that under different soil layers where confined water and suspended water and groundwater exist, in different seepage directions (seepage inside and outside the foundation pit) and under negative or positive excess pore water pressures, seepage force will have a great influence on the water and earth pressures. At the same time, it shows that in most 2-D seepage conditions, it is rather not to estimate the earth pressure with Rankine's theory but with graphic method of Coulomb's theory by searching the possible slip surface.

INTRODUCTION

The design and construction of deep foundation pit are often dominated by the flow of water in soil around the sheet piles. The seepage flow influences the stability of the excavation where seepage failure may occur. While the piping takes place at the excavation level, the heaving is more catastrophic and its risk is usually evaluated by considered a rectangular failure mechanism adjacent to the wall (Terzaghi, 1943). The vertical force equilibrium of this soil mass is then considered by neglecting the vertical frictional forces along the vertical faces of this mechanism. It should be noted that it is difficult to calculate the earth pressures when there is water in soils, especially when there is water with different being patterns, such as groundwater、 confined water and suspended water.

Based on laboratory model tests, Kastner(1982) has shown that the failure of the sheet piling structures in the presence of seepage flow is not only due to the heaving phenomenon but may also due to the reduction of the passive earth pressures in front of the wall. In some simple cases, such as the case of the single sheet pile driven into a semi-infinite homogeneous soil medium, the pore water pressure distribution is given analytically (Soubra & Kastner, 1992). Soubra, A. et al. (1999) used the variational limit equilibrium method to study passive earth pressure in the presence of hydraulic gradient, and verified that the shape of sliding surface at failure is a log-spiral. Lysmer (1970) was

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the first to use finite element and linear programming for limit analysis computations in soils. Hird et al. (2000) & Lin-Pei Choo et al. (2000) performed model study and analysis of water flow in multilayer soils. Wei RuLong (1998) & Chen Yujiong et al. (2000) paid more attention to the influence of seepage on earth pressures. In calculation of earth pressure on supporting structure, Li Guangxin (2000) discussed the methods for estimating the water and earth pressures separately and together on the supporting structure around a foundation pit.

The most difficult problem in estimating earth pressures is estimating the influence of water. Rankine's earth theory and Coulomb's earth theory are widely used for calculating earth pressures. The objectives of this paper are (1) compare and analyze the influence of seepage on the water and earth pressures in several different seepage conditions. (2) compare and analyze the applicability of Rankine's theory and Coulomb's theory in different seepage conditions.

WATER AND EARTH PRESSURES OF 1-D SEEPAGE

Referred to Zhang Zaiming et al. (2001), there are usually three patterns of water in Beijing. They are groundwater、confined water and suspended water. They exist in several different soil layers, which determine water and earth pressures on supporting structure, and the pressures may be different from that when the water is static. In the following cases, one- dimensional (1-D) seepage flow was considered in layers behind retaining structures, and the water and earth pressures were calculated and analyzed.

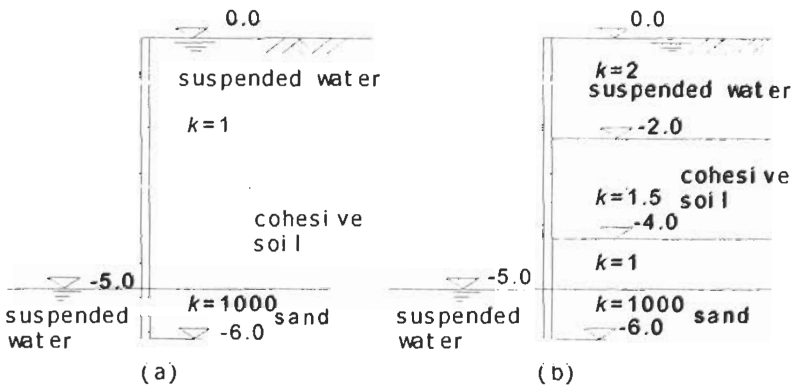


Fig 1.1-D seepage of different soil layers (example 1)

Example 1. Fig 1 shows two cases of retaining structures of different layers. There are two layers of soil as shown in Fig.1(a) and four layers in Fig.1(b), and seepage is vertical downward through every soil layer, and the coefficients of permeability k are relative values. In every soil layer, the saturated unit weight γ_{sat} of soils all is 18.5 kN/m^3, the angle of internal friction φ' is 30^0, and the cohesion c' of sand is zero and that of cohesive soil is 5kPa.

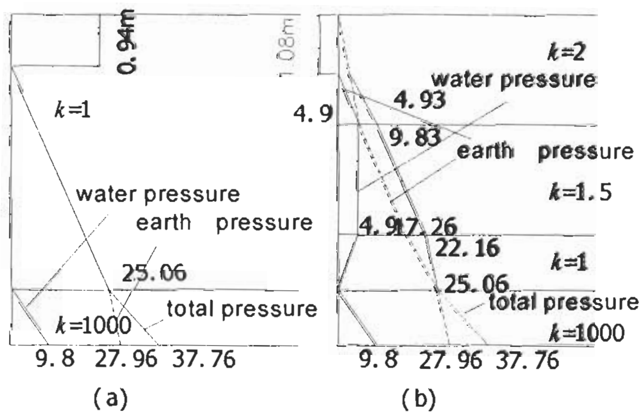


Fig 2.Water and earth pressure of different soil layers(example 1)

Considering seepage force, water

and earth pressures with depth are calculated with Rankine's earth pressure theory and shown in Fig.2 (i.e., earth pressure, water pressure and pressure applied on the retaining structure).

As shown in Fig.2 (a), the total pressures are obtained by calculating water and earth pressures separately in saturated clay silt. It is identical with that of calculating water and earth pressures together ($p_a = k_a \gamma_{sat} z$). As shown in Fig.2 (b), the distribution of different coefficients of permeability in soils causes different distribution of water and earth pressures. It is obvious that in case of the suspended water in soil layers, it is improper to calculate water and earth pressures separately in which earth pressure is calculated with submerged unit weight γ', and static water pressure is added to.

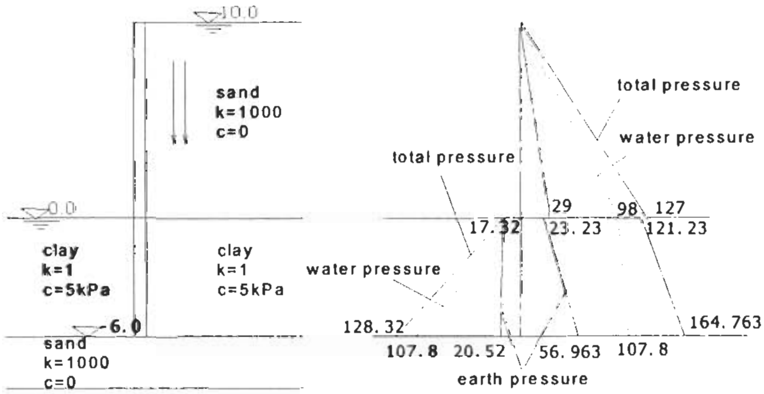


Fig 3.1-D distribution of water and earth pressures

Example 2. As shown in Fig.3, in every layer, the saturated unit weight γ_{sat} of soils is 18.5 kN/ m^3, the effective angle of internal friction φ' is 30^0, the effective cohesion c' of the clay is 5kPa, and the coefficients of permeability k are relative values. Because the coefficient of permeability in sand is much larger than that of the clay and confined water exists in the underlying sand layer, the calculation can be approximately considered as vertical downward and upward seepage along the sheeted wall. It can be seen from Fig.3 that in this case, the passive earth pressure in clay will remarkably reduce, which easily lead to seepage failure in the passive side ($(\gamma' - i\gamma_w)/\gamma_w$=0.533) and failure of the foundation pit.

Water and Earth Pressures in 2-D Seepage

In example 3, the condition of a foundation pit is shown as Fig.4. The soil is normally consolidated. The saturated unit weight γ_{sat} is 18.5 kN/m^3, the angle of internal friction φ is 30^0, the cohesion c of the clay is zero, and water head $\triangle H$ is 10m. The pressures are calculated with different methods, the resultant pressure E_1 on the left wall every meter (=Passive earth pressure P_P + water pressure E_{W1}) and E_2 on the right wall every meter(=Active earth pressure P_a + water pressure E_{w2}).

The pressures are calculated as follows:

(1) Method I. Earth pressure is calculated with Rankine's earth pressure theory, and the frictional force among sheet pile and soil is ignored. Hydraulic gradient and seepage force are supposed to be constant along the sheet pile. Water and earth pressures are calculated.

(2)Method II. Earth pressure is calculated with Rankine's earth pressure theory, but the seepage forces are calculated according to the flow net. The flow net is calculated with finite element programs and streamlines are drawn manually.

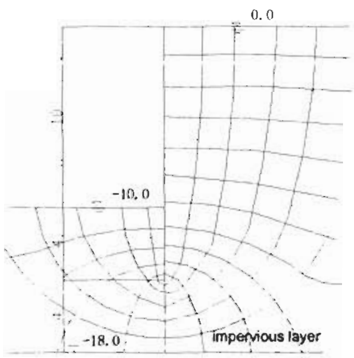


Fig.4. Diagram of sheet pile wall flow net

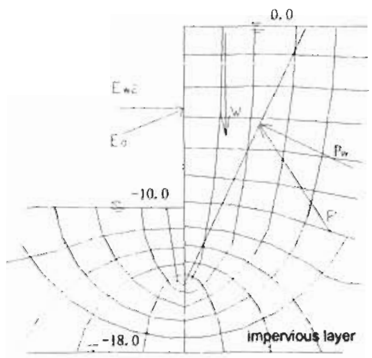


Fig.5. Active earth pressure of sheet pile wall

(3)Method III. Friction between the sheet pile wall and the soil is ignored, the resultant pressure on sheet pile wall is calculated with Rankine's earth pressure theory straightly adopting saturated unit weight, without any water pressure; namely, estimating the water and earth pressure together.

(4)Method IV. The sliding surface is calculated with Coulomb's earth pressure theory. Sliding surface is a plane as shown in Fig.5. On the right side of the sheet pile a slide wedge (including water and earth) is taken, and the weight of the slide wedge is calculated with saturated unit weight γ_{sat}. On the sliding surface there are supporting forces of R and water pressure P_W perpendicular to the sliding surface. P_W is calculated according to sections of the flow net cut by the sliding surface. The force E acted on slide wedge by sheet pile includes water pressure E_w and earth pressure E_a (or E_p).

The results of calculations with above-mentioned four methods are shown in Table 1. The total pressures are the resultants of the water pressure and soil pressure, and the angles of friction between the sheet pile wall and soil of method IV(2) and method IV(1) are 20^0, 0^0 respectively. It should be noted that in the FE programs, the seepage zone is discretized by a triangle mesh, and the soil is supposed to be isotropic and saturated.

The active and passive planes of slip in method IV(1) are inclined at are 35^0 and 60^0 to sheet pile plane, respectively; and those in method IV(2) are at 37^0 and 60^0. In method IV(2), the angles between outside normal of the sheet pile and resultant active pressure(including water pressure) and that between outside normal of the sheet pile wall and resultant passive pressure are 7.6^0, 10.1^0 respectively. If method IV(2) is taken as a standard, it can be seen from table 1 that when considering the friction between sheet pile wall and soils, the errors of the resultant active pressure and resultant passive pressure calculated with method III are relatively great, more than 1/3, indicating that it is not suitable to calculate water and earth pressures together with Rankine's earth pressure theory. The errors of the resultant passive pressures calculated with method I and method II are relatively great, nearly to 1/3; and those of resultant active pressure are small. Comparing the calculations of method IV(1) and those of method IV(2), it can be seen that friction's influence on resultant active pressure is small in spite of different directions while the influence on resultant passive is great. It should be noted that when friction δ between sheeted pile wall and soil is relatively great; the resultant passive earth pressure calculated with Coulomb's theory of method IV(2) is too large.

Table 1. Earth pressure on the sheet pile wall (example 3)

Methods		Earth pressure		Water pressure		Resultant pressure	
		Left side P_p /kN	Right side P_a /kN	Left side E_{W1} /kN	Right side E_{w2} /kN	$(\bar{P}_p+\bar{E}_{W1})$ /kN	$(\bar{P}_a+\bar{E}_{w2})$ /kN
Method I		79.44	460.27	122.304	422.576	201.74	882.85
Method II		48.63	422.494	153.36	579.345	201.99	989.81
Method III		-	-	-	-	444	604.333
Method IV1)		54.29	400.47	153.36	579.345	200.61	971.82
Method IV(2)		156.33	353.54	153.36	579.345	304.99	919.55
Compare with method IV(2) /%	Method I	-49.2	30.2	-20.3	-27.1	-33.9	-4.0
	Method II	-68.9	19.5	0	0	-33.8	7.6
	Method III	-	-	-	-	45.6	-34.3
	Method IV (1)	-68.3	15.1	0	0	-34.2	5.7

COMPUTATIONS OF WATER AND EARTH PRESSURES IN DEWARTERING

When groundwater level is very high, dewatering is often needed, consequently the water and earth pressures will be different when different dewatering measures are taken. In the section, water and earth pressures are calculated and analyzed with such two dewatering methods respectively; pumping from sumps and well-point dewatering outside the foundation pit.

Example 4. The foundation pit is 5 meters deep, and the sheet pile wall is 7 meters deep. The dewatering is performed by well point system. It is supposed that the water levels are at ground level inside and outside foundation pit respectively. The impermeable stratum is 13m deep from original ground level. Riser is 7m deep, 2m lower than foundation pit. The soil in foundation pit mainly is normally consolidated, the saturated unit weight γ_{sat} of soil is 18.5 kN/m^3, the angle of internal friction φ' is 30^0, the cohesion c' of the clay is zero, and the friction angle δ between soils and sheet pile is 18^0. Water levels inside and outside the foundation pit are supposed to be constant. According to some references (Zhang Yunbo, Shun xinzhong (2000) and Mao Changxi (1990)), equipotential lines in the vacuum zone and the gravitational zone are drawn with finite element method shown as Fig.6. In the FE programs, the seepage zone is discretized by a triangle mesh, and the soil is supposed to be isotropic and saturated.

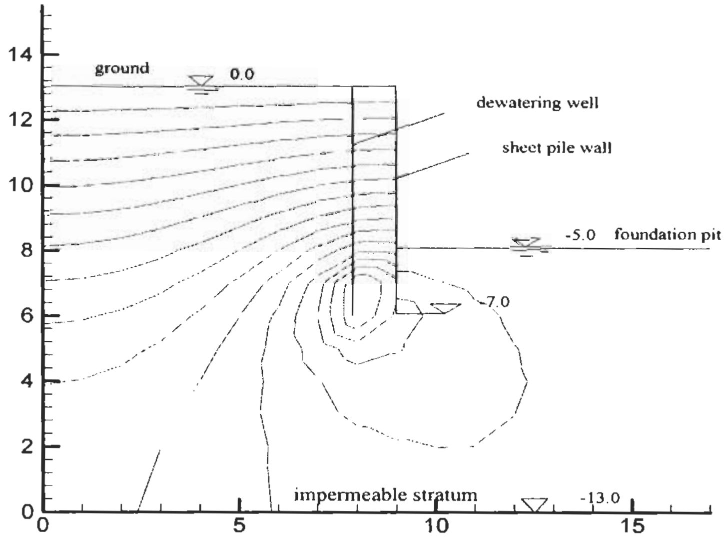


Fig.6 Equiptential lines diagram of well-dewatering flow net

Calculated with method IV(2) above, water pressures of the sheet pile are E_{W1} and E_{w2}, and the resultant pressures of that are $\overline{E}_a$ and $\overline{E}_P$ respectively.

If pumping from sumps is used in the same project. Water levels inside and outside the foundation pit are supposed to be constant, water and earth pressures are calculated and analyzed with finite element method and method IV(2) above. Water and earth pressures are also calculated with Coulomb's earth theory on condition that there is no seepage. The results of computations are shown as Table 2.

Table 2. Earth pressures of well-dewatering and open-channel drainage (example 4)

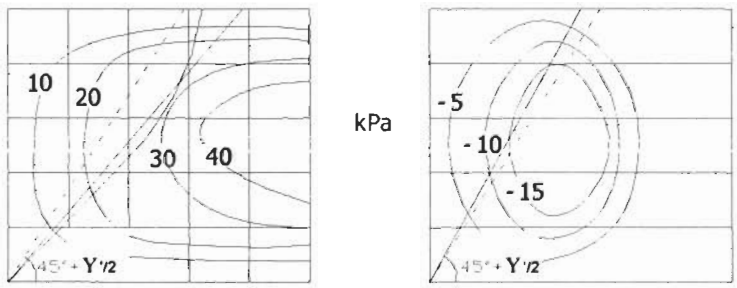
Method		Earth pressure		Water pressure		Resultant pressure	
		P_a /kN	P_P /kN	E_{W1} /kN	E_{W2} /kN	$\overline{E}_a$ /kN	$\overline{E}_P$ /kN
Dewatering outside		136.94	139.55	7.82	8.51	144.40	147.67
Dewatering inside		92.76	47.02	148.04	22.10	237.99	68.38
No seepage		61.74	94.39	240.1	19.6	299.43	113.19
Comparison with Dewatering inside (%)	Outside	-32.3	-66.3	1793.1	159.7	64.8	-53.7
	No seepage	-54.9	-32.4	2970.3	130.3	107.4	-23.3

It can be seen from table 2 that, because of seepage, well-dewatering outside the foundation pit can reduce resultant active water and earth pressure, and increase resultant passive earth pressure, which is favorable to slope stabilization of foundation pit. From the results calculated with Coulomb's theory it can be seen that, because of seepage, the inclination angle of slip surface in well-dewatering outside is 38^0 which is larger than $45^0-\varphi/2=30^0$, and that of the passive is 72^0 which is larger than $45^0+\varphi/2=60^0$. The inclination angle of slip surface of earth wedge in pumping inside the foundation pit is 38^0, and that of passive is 68^0, when there is no seepage (viz. static water), that of active is 33^0, and that of passive is 70^0. It illuminates that in method IV(2), seepage affects obviously to the inclination angles of slip wedge both in active side and passive side.

WATER AND EARTH PRESSURES OF 2-D SEEPAGE WITH EXCESS PORE WATER PRESSURE

If the fill behind the retaining wall is cohesive soils, excess pore water pressure often appears in the fill when the soil is completely saturated or highly saturated. The excess pore water pressure is generally non-linear distribution along the retaining wall; when the retaining wall is permeable, the seepage will be two dimensional (2-D) and change as time goes. In this case, it's not suitable to calculate earth pressure with Rankine's earth pressure theory, but with the graphic method of Coulomb's theory.

Example 5. The fill behind a 5-meter deep retaining wall is cohesive soil, and both the ground surface and underlying stratum are pervious. The degree of saturation of fill S_r = 90%, the unit weight $\gamma = 20$ kN/m^3, the coefficient of permeability $k = 1.0 \times 10^{-6}$ cm/s, the effective cohesion intercept c' = 5kPa, effective angle of internal fiction φ' = 30^0, the initial void ratio e_0 = 0.5, and the coefficient of compaction a = 0.2MPa^{-1}. It divides into ten layers and each layer is built in 8 hours. After filling, a uniform surcharge q = 20kPa is applied equably on the fill surface. The distribution of pore water pressure in the fill after 24 hours is calculated and shown as Fig.7. This 2-D seepage problem should be calculated with the graphic method of Coulomb's theory, and the inclination angle of sliding surface is 37^0(not 45^0-$\varphi'/2$ = 30^0) in positive pore water pressure. The distribution of excess pore water pressure u、the effective active pressure on the retaining wall p_a' and water and earth pressure p_a are shown in Fig.8. The resultant pressure of active state E_a is 155 kN in positive pore water pressure (when there is no pore water pressure E_a is 104.5 kN). It is to be noted here that u is the water pressure on sliding surface, and the water pressure on the retaining wall is zero.



(a) positive pore water pressure of 2-D

(b) negative pore water pressure of 2-D

Fig.7. 2-D ditribution of the excess pore water pressure

Example 6. The soil、 uniform surcharge and boundary conditions are the same as example 5, and now the problem is of supporting structure when the foundation pit is excavated. Because of excavation of the original state soil、 unloading and the displacement of supporting structure, the negative excess pore water pressure is supposed to be distributed as Fig.7 (b). The earth and water pressures should be calculated with the graphic method of Coulomb's theory. The inclination angle of sliding surface is 62.6^0 to the surface, and the corresponding resultant active pressure on the supporting structure E_a is 78.5kPa.

The negative excess pore water pressure on the sliding surface、 effective active earth pressure and resultant active pressure(water and earth pressure) are shown in Fig. 9.

It can be seen from the two examples:

a) Because of negative excess pore water pressure, the resultant water and earth pressure acted on the bracing wall deduces greatly. The influence of negative pore water pressure Δu can not be reflected by strength parameters of conventional undrained triaxial test.
b) In the examples the degree of saturation is relatively high and not complete saturation, and in case of complete saturation, water pressure should be calculated and applied on the wall. This can not be reflected in calculating water and earth pressure together.
c) In the calculations above, it is the excess pore water pressure on the sliding surface not that on the retaining wall that determines the resultant pressure on the retaining wall.

CONCLUSIONS

1) When there exist suspended water, the computations of water and earth pressures are quite different from those of hydrostatic pressure. When the coefficients of permeability of the soil layers are quite different and confined water may exist, the passive earth pressure will remarkably reduce, even lead to seepage failure in the passive side, which is unfavorable to the stability of the foundation pit.

2) In comparison with drainage inside foundation pit, dewatering with well-point outside can reduce active earth pressure and increase passive earth pressure, which is favorable to slope stabilization of foundation pit.

3) When there is 2-d distribution of excess pore water pressure, it should be calculated with the graphic method of Coulomb's theory, not Rankine's theory. According to calculations with Coulomb's theory, the inclination angle of sliding surface is not $(45^0 - \varphi'/2)$; if there is positive pore water pressure, the sliding surface is gentler; while there is negative pore water pressure, the sliding surface is steeper. And the real sliding surface is likely to be curve surface, which is close to the plane surface calculated with the

graphic method of Coulomb's theory. The sliding surface is the best to cut the positive pore water pressure zone and escape from the negative pore water pressure zone.

4) In the calculations above, sometimes it is the excess pore water pressure on the sliding surface not one on the retaining wall that determines the resultant pressure on the retaining wall.

5) Rankine's earth pressure theory is suitable to 1-D seepage, but not to 2-D seepage. The computation with flow net and Coulomb's theory is more reasonable.

REFERENCES:

Chen Yujiong, Wen Yanfeng. "Water and earth pressures on the supporting structure around a foundation pit." *Chinese Journal of Geotechnical Engineering* (in Chinese), 1999, 21(2): 139-143.

Hird, C.C., Moseley, V.J. (2000). "Model study of seepage in smear zone around vertical drains in layered soil." Géotchnique, ICE, 50(1), 89-97.

Kastner, R. (1982). "Excavations profondes en site urbain: problemse liés à la mise hors d'eau. Dimensionnemeni des soutènements butonnès. Theses Sciences," INSA Lyon and University Claude Bernard.

Li Guangxin. "Estimating the water and earth pressures on the supporting structure around a foundation pit separately and together." *Chinese Journal of Geotechnical Engineering* (in Chinese), 2000, 22(3): 348-352.

Lin-Pei, Choo, Ernest K. Yanful. (2000). "Water flow through cover soil using modeling and experimental methods." *Journal of Geotechnical & Geoenvironmental Engineering*, ASCE, April 2000.

Lysmer, J. (1970)."Limit analysis of plane problems in soil mechanics." *J. Soil Mech. And Found. Div.*, ASCE, 96(4), 1131-1334.

Mao Changxi(1990). *Analysis and Control of Seepage Computation* (in Chinese). Beijing: Hydraulic and Electric Press, 195~196.

Soubra, A.-H, Kastner, R.(1992). "Influence of seepage flow on the passive earth pressures." *Proceedings of the international conference on the retaining structures* (ed. C.R.Clayton), pp.67-76, Cambridge:Thomas Telford.

Soubra, A.-H, Kastner, R. and Benmansour, A.(1999). "Passive earth pressures in the presence of hydraulic gradients." Géotechnique, ICE, 49(3), 319-330.

Terzaghi, K. (1943). *Theoretical Soil Mechanics*. New York:wiley.

Wei RuLong. "The computation of earth pressure of deep sheeted excavations." *Ground Improvement* (in Chinese), 1998.9(1): 3-15.

Zhang Yunbo, Shun Xinzhong(2000). *Dewatering Projects of Foundation Pit* (in Chinese). Beijing: Earthquake Press, 26~34.

Zhang Zaiming, Sun Baowei, Xu Hongsheng. "Effect of characteristics of ground water distribution and seepage on anti-uplift analysis of building foundations." *China Civil Engineering Journal* (in Chinese), 2001, 34(1).

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冻结工法在上海地铁建设中的应用与发展

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Application and Development of Freezing Method in the Construction of Shanghai Metro

Xiao Cheng
Shanghai Metro Construction Corporation Limited

Summary

This paper describes the theory, research and technology application of freezing method in the present Shanghai Metro construction. Example is given of the tunnel shield construction between Pudong Main Road Station and Yangshupu Station of Shanghai Metro Pearl Line Phase II development. It illustrates the application advantage in the use of freezing method to successfully improve the ground around the tunnels. Finally, given the major development opportunities facing the rail transportation today, it analyses the wide application prospect of freezing method and proposes the future direction in research development.

Application and Development of Freezing Method in the Construction of Shanghai Metro

Cheng Xiao[1]

ABSTRACT: This paper summarizes the present status of both the theoretical research and the construction method of the freezing method as used in Shanghai metro construction. It analyzes the advantages of applying this method, which are exemplified by the successful application thereof in the consolidation of the soil body at the shield machine exit of the tunnel section between Pudong Avenue Station and Yang Shu Pu Road Station of the Shanghai Metro Pearl Line, Phase II project. Finally, facing the opportunity of great developments in the metro construction in Shanghai, it analyzes the broad prospect of applying this method and gives comments on the research orientation thereof in future.

INTRODUCTION

Mr. F.H. Poetch, a German engineer in 1880, first proposed the principle of artificial freezing method. This method was successfully applied to the pit shaft construction in a coalmine in Albari in Germany in 1883. Since then, this artificial soil layer freezing technology has been extensively applied and has gradually become the main construction technology of shaft engineering projects in various countries around the world. It has been more than forty years since this freezing method was first applied to the shaft sinking construction in coalmines in our country. China is among those countries where the shaft sinking method is used cut through the thickest layer of topsoil.

In recent years, thanks to the rapid development of the urban rapid transit and with the continuous progress of the freezing theories, people have started to apply this artificial soil layer freezing technology to the urban underground construction and this technology has become more and more mature. This article is mainly focused on introducing the application of this method in the soil body consolidation at both the entrances and exits for the river-crossing shield machines of Shanghai Metro Pearl Line, Phase II project. It also anticipates briefly a broad prospect of both the application and development thereof in the Shanghai metro construction.

FREEZING THEORY & PROCESS

Study on the Freezing Mechanism

By utilizing the artificial freezing technology, the freezing method enables the water in the soil layer to ice up and enables the loose water-bearing rock-soil to become frozen soil thereby isolating the underground water. It is a physical consolidating method of temporary support by relying on the strength and stability of the consolidated soil body.

The caloric migration and the frost heaving are two matters of the utmost concern in the freezing process. It is generally believed that the migration of the water content is in conformity with the theory for the migration of the membrane water in the soil (See Figure 1) and that the frost heaving is mainly caused by the following two reasons: 1) a 9% increase of volume after the water in the frozen soil has become ice; 2) the ice crystal either on the frozen surface or formed inside the frozen soil due to the water content migration.

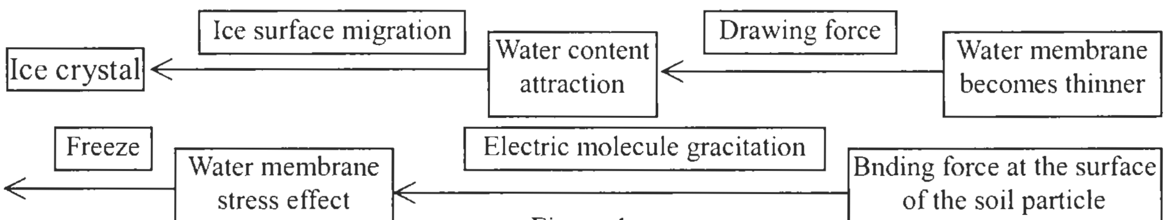

Figure 1

[1] CEO and professorial senior engineer of Shanghai Metro Construction Corporation, No.12,Heng-shan Road, Shanghai, China.(Zip code:200031).

Mechanical Properties of Frozen Soil

Generally speaking, the soft clay in the Shanghai region has the following main mechanical properties:

(1)The strength of the frozen soil is in a linear relation with the negative (below zero) temperature thereof. The lower the temperature is, the higher the strength will be. However, the strength of the sludge-natured clay will no longer increase when the temperature thereof is lower than -80°C. The strength of the frozen soil is also related to the gradient of the temperature drop. In case of rapid temperature drop, the water in the rock-soil may easily forms the hexahedral ice crystal and consequently the strength of the frozen soil is high;

(2) Studies show that both the density of the soil body and the strength of the frozen soil decrease with the increase of the water content in the sludge-natured clay;

(3) Creepage is one of the obvious features of frozen soils. The creep age curve can be divided into both an elastic stage and a creep deformation stage. The latter may be described approximately in a form of negative powers in accordance with the change of time;

(4) Mr. Ma Wei analyzed the stress-strain curves of frozen soils for different stress routes and got apparently different results thereof. The yield strength along the load decreasing stress route is obviously less than that along the load increasing stress route. The difference between the two is getting larger and larger with the increase of the surrounding pressure and the decrease of the temperature;

(5) The deformation of the frozen soil varies under different temperature drop conditions. The freeze expansion feature of the non-homogeneous clay is characterized by its heterogeneity. The freeze expansion process is also accompanied by the phenomenon that the frozen soil increases the density of the non-frozen soil by means of its pressure, i.e., cold-induced concretion.

Freezing Construction Process

At present, two kinds of freezing processes have been utilized in the Shanghai region, i.e., ammonia-salty water freezing and liquid nitrogen freezing. The ammonia-salty water freezing method is accomplished by replying on freezing technology based on nitrogen circulation. As a cooling medium, the salty water transfer the heat from the upper layers to the cooling water and finally the heat is released to the atmosphere via the cooling water in circulation. Ammonia, salty water and cooling water, the three big circulation systems constitute the heat pump. The cooling system is arranged as shown in Figure 2.

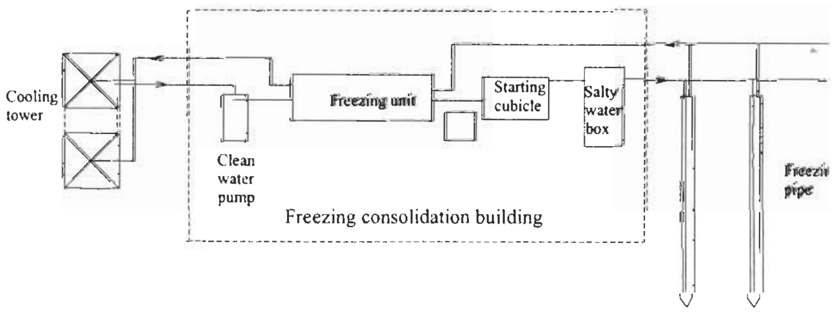

Figure 2 - Sketch of Arrangement of Freezing Station & Freezing System

Owing to its technical and economical features, the ammonia-salty water freeze method is extensively used in the soil body consolidation of both the entrances and exits of metro tunnels, the horizontal freezing of passageways, the protection of pipelines covered by shallow soil layers and a large amount of other projects.

Liquid nitrogen freezing (Figure 3) utilizes one feature that the vaporization temperature of the industrially used liquid nitrogen is –195.8°C. The low-temperature liquid nitrogen exchanges heat directly with the soil layer to be frozen thereby cooling the soil layer. In this process, the liquid nitrogen is refrigerant and cooling medium at once. The temperature of the liquid nitrogen is low, the amount of heat exchanged is great and the cooling speed is fast. Owing to all of these, this method is often utilized in emergency rescuing operations where there are rich sources of underground water and the soils have strong theological properties. It is an environmentally hygienic and technically safe measure for rock-soil engineering projects.

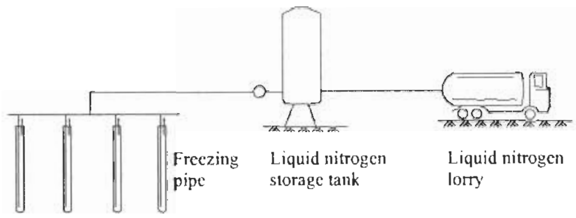

Figure 3 - Liquid Nitrogen Freezing System

APPLICATION EXAMPLES IN ENGINEERING PROJECTS

Brief of Engineering Project

For the river-crossing tunnel section between Pudong Avenue Station and Yang Shu Pu Rd. Station of Shanghai Metro Pearl Line, Phase II project, the shield machine is used in the construction. The shield machine starts from the Pudong Avenue.

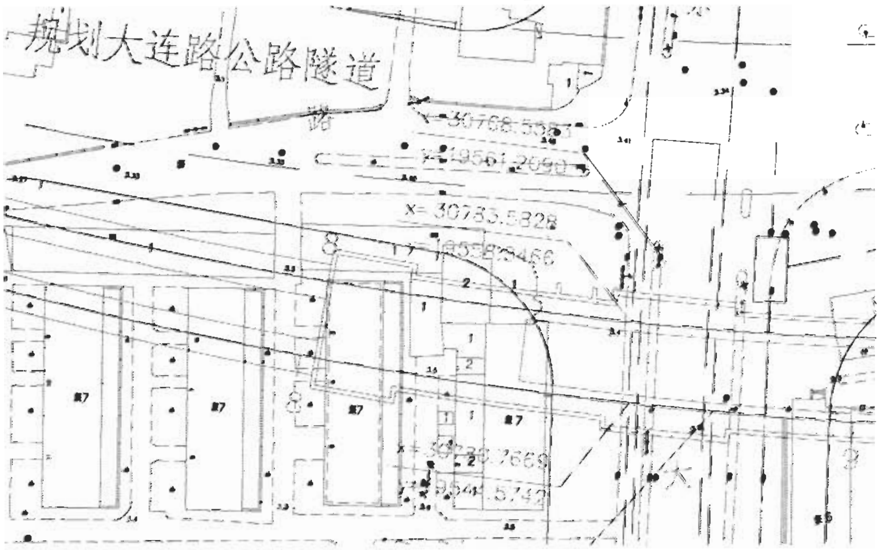

Figure 4 - Arrangement Plan of Shield Exit Area

The soil layer conditions for this tunnel section are very complicated. The soil layers are distributed in proper order as follows: fill, gray sludge-natured and dust-natured clay, gray sludge-natured clay, gray clay and gray dust-natured clay, etc. The diameter of the shield exit is 6.7m. The depth of the main body structure of the tunnel is 16m. The diaphragm wall with the thickness of 0.8m is used to surround and protect the shield shaft. The northern side of this tunnel section is immediately adjacent to the Da Lian Rd. River-crossing Tunnel under construction at the same time. Moreover, at about 7m directly in front of the shield exit, there is a 7-floored building that needs to be protected (It is only 4m away from the diaphragm wall.). Therefore, the construction site is very much limited in space and the building protection requirement is stringent.

Freezing & Consolidating Scenario

It is necessary to consolidate the soil body at the shield exits in order to ensure the safety of the shield coming out of the exit. A comparison of the normally used consolidating scenarios is made and shown in Table 1. Whereas both the installation and the construction of the shield machine are carried out at the same time and in order to minimize the impact on the ambient environment, a vertical local freezing consolidation scenario is finally adopted. In accordance with the calculation, the consolidation range is determined as follows: Transversely, it exceeds both the left and the right extremity of the cross section of the tunnel by 1.85m, respectively; vertically, it exceeds both the top and the bottom extremity of the cross section of the tunnel by 3m, respectively; and longitudinally, the thickness of this freezing consolidation body is 1.85m. The freezing pipelines are arranged as shown in Figures 5 & 6. The main freezing parameters are shown in Table 2.

Table 1

	Partial Freezing	SMW Method	Grouting (Rotating Grouting)	Mixing Pile
Consolidation Area	Small	Small	Relatively Small	Large
Construction Area	Small	Large	Relatively Small	Large
Strength & Homogeneity of Consolidated Soil Body	High Strength & Homogeneous Consolidation	Pulling out steel profiles involves risks.	Relatively High Strength & Relatively Homogeneous Consolidation	Relatively Low Strength & Homogeneous Consolidation
Slurry Pollution	No	Little	Serious	Little
Resulted Deformation of Ambient Soil Layers	Small	Small	Relatively Large	Relatively Large
Consolidation Duration	1.5 Months without Occupying the Construction Period (It can be implemented together with the shield installation.)	1 Month Occupying the Construction Period	1.5 Months Occupying the Construction Period	1.5 Months Occupying the Construction Period
Expense	High	Relatively High	Relatively High	Relatively High

Table 2

Construction Data	Quantity	Construction Data	Quantity
Freezing Depth	6.2~19m	Distance between Freezing Holes & Slot Walls	0.2m
Average Temperature of Frozen Soil Wall	-10℃	Allowable Inclination of Freezing Holes	3%
Active Freezing Time	25 Days	Lowest Temperature of Salty Water Required by Design	-25~28℃
No. of Freezing Holes	39	Salty Water Flow for Each Hole	$6m^3/h$
Distance between Two Adjacent Freezing Holes	Distance between Two Adjacent Rows of Holes: 0.7m; Distance between Holes: 0.8m	Temperature Detecting Holes & Pressure Detecting Holes	5

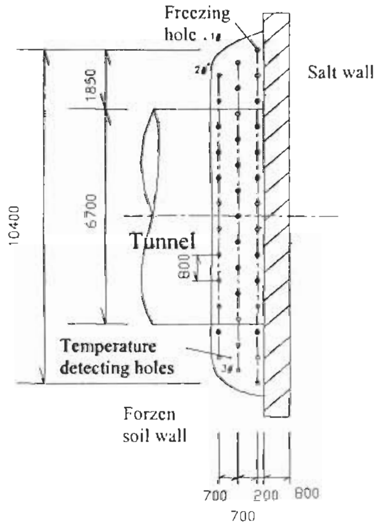

Figure 4 - Plan of Freezing Hole Arrangement

Figure 5 - Freezing Construction Site

Analysis of Freezing Effect

Development of Cooling Temperature

The cooling station goes into operation in the actual construction. The temperature of the salty water drops to -22°C five days later and is kept at –33.5°C 22 days later. Owing to the relatively fast temperature-dropping gradient and owing to the relatively low salty water temperature, the speed of the frost soil development is expedited. The speed of the frost soil development at the early stage reaches 70mm/d and the average speed of the frost soil development is 40mm/d. 25 days after the freezing, the freezing consolidation proves to meet the design requirement through the inspection.

Analysis of Freeze Expansion Pressure

The freeze expansion pressure resulting from the freezing is one of the main factors affecting the deformation of soil layers. The following conclusion can be reached after analyzing the freeze expansion pressure under the engineering conditions for this case:

(1) The impact range of the freeze expansion of the frozen soil exceeds transversely both the left and the right extremity by less than 1.06m, respectively and it exceeds the top extremity by less than 1.2m;

(2) The freeze expansion pressure is much greater than that of the water and soil in their original status. Especially, the pressure within the frozen soil layer rises up to 3~4 times the original one;

(3) The freeze expansion pressure remains basically the same with the change of the depth;

(4) Partial freezing technologies can effectively reduce the freeze expansion pressure. The maximum freeze expansion force measured at this time is 0.632MPa. The maximum freeze expansion force for similar projects (such as the East Yan An Road Tunnel) is normally about 2 MPa.

Control of Ground Subsidence

The construction of the adjacent Da Lian Road Tunnel is carried out at the same time. The 7-floor building to be protected has already subsided up to 2 cm. Owing to the joint impacts of the boring of shield machines, the melting subsidence of the frozen soil body and the pit excavation of the Da Lian Road Tunnel, the safety of the said building can hardly be ensured without taking any special measures. The following measures are taken in the construction:

(1)Fill back the cement-clay slurry immediately after pulling out the freezing pipes so as to consolidate the freezing hole;

(2)Implement the follow-up grouting for the buildings and places that may be affected by freezing(Figure 6). A total of 38 grouting holes are arranged in several rows. The grouting is made in time in accordance with the monitored situations. The total volume of all the grouted single-liquid slurry is 25 m^3 and that of all the grouted double-liquid slurry is 14.2 m^3;

(3)The grouting for the shield machine is carried out at the same time and the volume of the grouting for each ring of lining segments is 3.5 m^3;

(4) Refill the double-liquid slurry behind the wall within the tunnel.

After 45 rings of lining segments are assembled, the shield machine passes the said building successfully. Meantime, owing to the appropriate grouting, the total accumulated ground subsidence decreases from the maximum 64mm to within 30mm thereby ensuring the safety of the building.

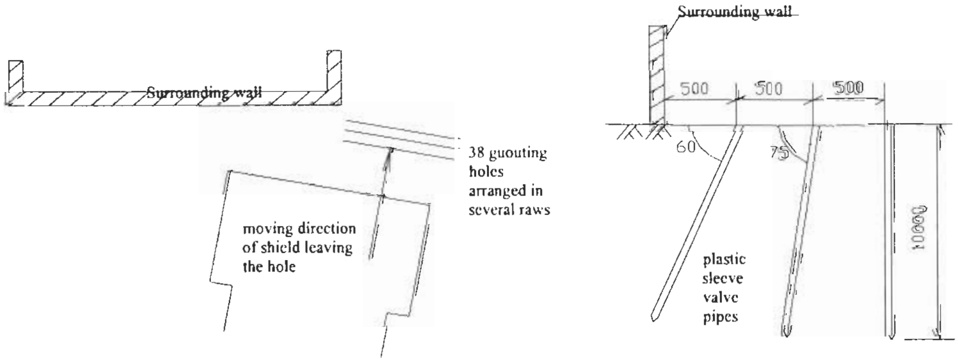

Figure 6 - Arrangement of Follow-up Grouting Pipes

ENGINEERING APPLICATIONS & RESEARCH PROSPECTS

In fact, the freeze consolidation has not only been successfully applied into the soil body consolidation of the entrances and exits for shield machines, it has also been extensively applied to the construction of both the passageway between adjacent metro tunnel sections and the pump building of metro tunnel sections. The horizontal freeze consolidation method is successfully applied to the construction for both the passageway between adjacent metro tunnel sections below the Huang

Pu River and the passageway linking Jing An Temple Station with West Nan Jing Road for Shanghai Metro Line No. 2. It is also successfully applied in connecting vertically the overlapping tunnel sections during the construction of the wind shaft at the river-crossing tunnel section for the Shanghai Metro Pearl Line, Phase II project under construction.

Besides, the Shanghai Stadium Station for the Shanghai Metro Pearl Line, Phase II project goes underneath part of Shanghai Metro Line No. 1. The freeze consolidation method is used to consolidate the soil body and the top down method is also used. Therefore, it set a precedent for applying the freeze consolidation method to the metro station construction and it will offer new inspirations for the construction of the metro transfer stations in future.

With the increase of application of the freeze consolidation method into the urban underground projects, more stringent requirements are raised for the freeze consolidating technologies. Further studies need to be carried out in terms of the following aspects regarding design theories:

(1) Studies on the mechanical performance for the frozen soft clay so as to offer more accurate bases for selecting calculation models as well as stress and strain indexes;

(2) Studies on the Sensitivity of the freeze consolidation as well as on the melting subsidence and the freeze expansion. Especially, it is necessary to take protective measures when there are stringent requirements on protecting the ambient environments;

(3) Studies on the dynamic performance of the frozen soil. It is very critical to protect the already completed line during the construction of the new urban rapid transit line especially when they are immediately adjacent to each other;

(4) Long-term impacts on the subsidence deformation after the freezing consolidation;

(5) Regarding design standards, at present, the equivalent relevant standards of the Coal Mining Ministry are basically referred to in terms of the freezing consolidation design. In view of the strict requirements of the urban underground projects, the relevant design and theoretical standards should be standardized.

CONCLUSION

It is not difficult to discern that there will be broad prospects for the application of the freeze consolidation method into the Shanghai metro construction with the increasingly stringent requirements for protecting ambient environments in future. Its unique advantages such as its reliable consolidating effects, good water resistance performance and flexible adaptability, etc. become more and more recognized. Consequently, the freeze consolidation design and the construction technologies will develop and progress continuously.

REFERENCES

Weng, Jia Jie, Etc., (1999), "Action of Water Content migration and Concretion", *Underground Projects and Tunnels*, 1999(1), 2-8.

Wang, Chong Xian, Etc., (2000), "A Tentative Study on the Mechanical Characteristics of the Clay in Shanghai under Negative Temperatures", *Underground Railway Corpus,* 380-384.

Cheng, Guo Dong, (2000), "New Progresses in International Studies on Frozen Soil Mechanics & Frozen Soil Projects".

Shanghai Tunnel Projects PLC, Etc., (1996), "Study on Freezing Consolidation Technologies for Shield Tunneling Construction".

Shanghai Tunnel Projects PLC, Etc., (2002), "Construction Organization and Design of the Freezing Consolidation for the Tunnel Section (between Pu Dong Avenue and Yang Shu Pu Road) of Shanghai Rapid Transit Pearl Line, Phase II Project".

Shanghai Tunnel Projects PLC, Etc., (2002), "Summary of Monitoring the Freezing Consolidation Pressure at the Shield Exit of the Pu Dong Avenue Station of the Pearl Line".

廣州地鐵一號線體育中心站基坑支護結構綜合測試研究

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Monitoring and Analysis on the Deep Excavation Retaining Structure of the Guangzhou Sport Center Subway Station

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撮要
本文給出了我們在廣州地鐵一號線體育中心站基坑支護結構綜合測試的成果和分析研究。該地鐵站基坑測試內容包括：支護樁鋼筋應力、測向位移、支撐受力和基坑施工過程中的周邊地下水位元的變化情況。基於測試結果，本文進行了綜合分析，並提出了切實可行的應用於實際工程的基坑支護優化方案，創造了便利施工條件，降低了工程造價、縮短了施工工期。

Monitoring and Analysis on the Deep Excavation Retaining Structure of the Guangzhou Sport Center Subway Station

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Abstract: This paper describes the integrated monitoring on the deep excavation retaining structure of the Guangzhou Sport Center Subway Station. The monitoring includes the stress of reinforced bars of the piles, lateral displacement of the piles, the strut force and water level during the construction. Based on the monitoring results, this paper gives the comprehensive analysis and proposes the optimized scheme of the retaining structure that was applied to decrease the total investment, reduce the time limit of the project and make the construction more convenient.

1. Introduction

The deep excavation of the Guangzhou Sport Center Subway Station is 260m long, 22m wide and 14~15m deep. The retaining structure of the deep excavation consists of the soldier piles and steel tube struts. The pile diameter is 1.2m and the distance between the centers of the neighboring piles is 1.35m.The external diameter of the steel tube struts is 600mm and the thickness of the tube wall is 12mm.The horizontal distance between the neighboring struts is 3m in the original design and changed to 4m according to the monitoring results. The profiles of the deep excavation are shown in fig.1.

The purpose of the monitoring is 1) to ensure the safety and economy of the project, 2) to promote the research on the typical type of retaining structure, 3) to check the theory and geological parameters of the design.

This work began in Dec. 1995 and ended in Apr. 1997.

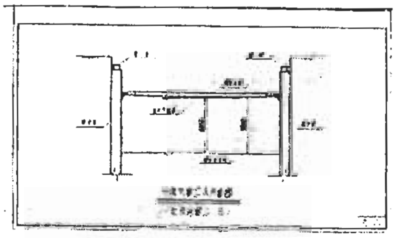

图 2 基坑北段支撑

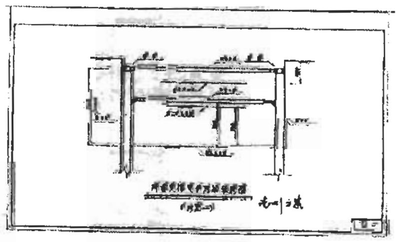

图 3 基坑南段支撑

Fig.1 Profiles of the retaining structure

2. Geological conditions

The engineering geological profiles consist of the following layers.

1) Artificial fill: The main composition of the artificial fill is clay and sand, with some gravel. The thickness of the layer is about 1~3m.
2) Sand: Fine and medium-sized, mediate dense, from wet to saturated, with coarse sand at the bottom, well graded. The thickness of the layer is about 2~5m.
3) Clay: From plastic to hard, uniformed, with some silty sand and silty clay. The thickness of the layer is about 1~4m.
4) Silty clay: Hard, non-uniformed, with some gravel, weathered from siltstone and sandstone. The thickness of the layer is about 1~6m.
5) Bedrock: Siltstone and sandstone, form strong weathered to little weathered.

The groundwater of the station is mainly pore water and fissure water. The fissure water is stored in the fracture zone. The alluvial-diluvial clay and residual soil are impervious layers.

3. Content and implement of the monitoring

The content of monitoring includes the following items:

1) Stress of the steel struts: This measuring method used dial gauges and a one meter long steel bar of the same material with the struts, which can eliminate the temperature stress. And 18 survey points were installed in 8 profiles.
2) Lateral displacement of the piles: The inclinometer was used to measure the deflection of the piles, with the casings attached to the reinforced bar in four holes of two profiles.
3) Stress of the reinforced bars of the piles: This measured items used the reinforced bar strain meters, which were installed in 16 measuring points of four bars in two piles of two profiles.
4) Groundwater levels: Measured with 6 water level tubes of which two were installed under the bottom of the foundation and four were installed beside the piles of two profiles.

The monitoring scheme is shown in fig.2 and the two main monitoring profiles are shown in fig.3 and fig.4.

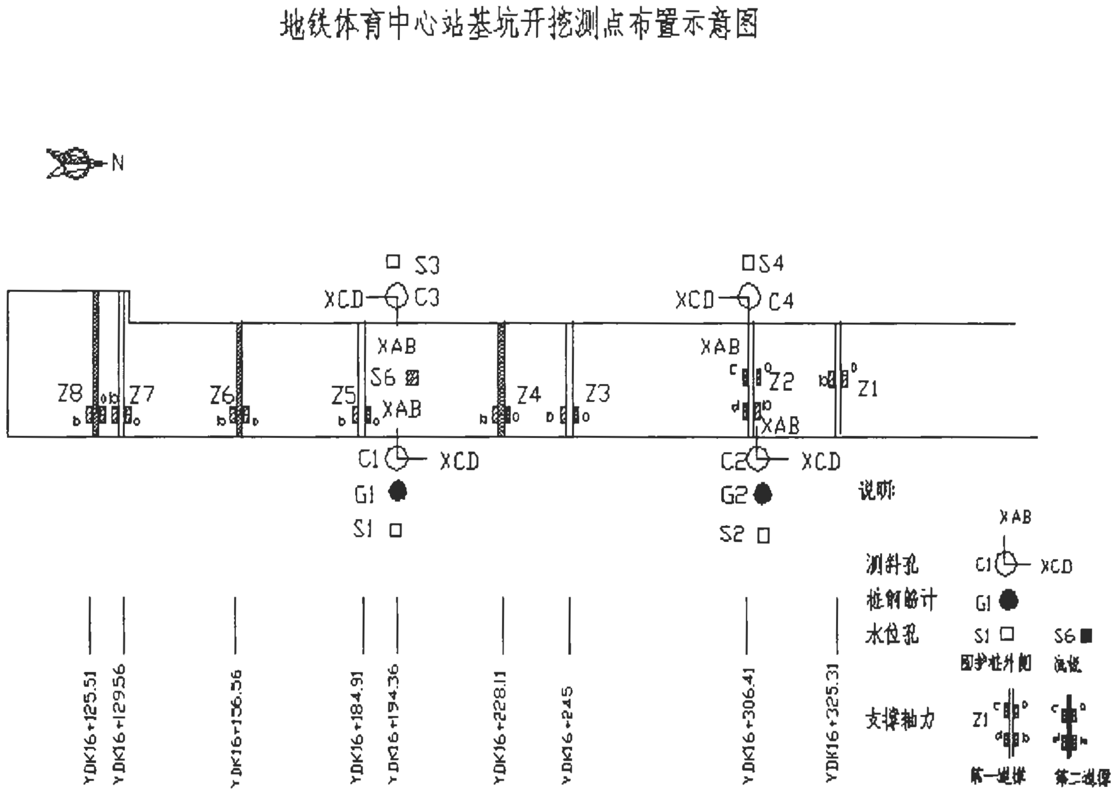

Fig.2 The plane scheme of the monitoring

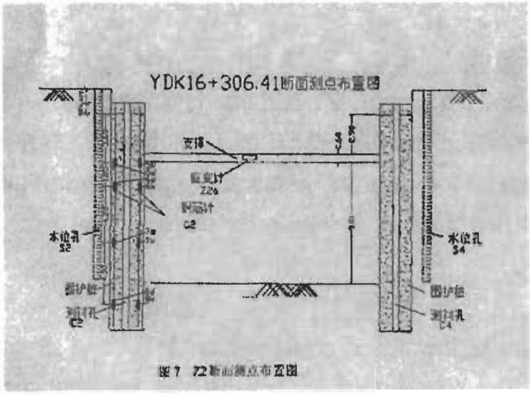

Fig.3 Monitoring profile Z2

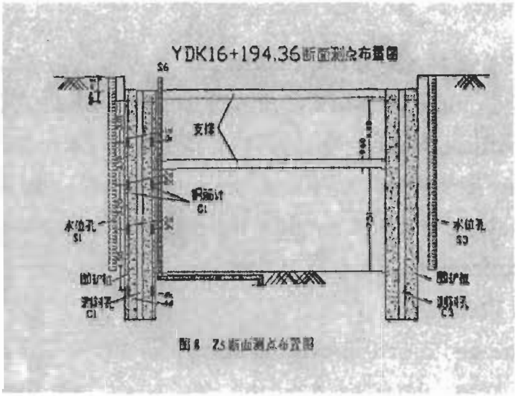

Fig.4 Monitoring profile Z5

The instruments should be installed simultaneously with or before the construction and excavation. The construction procedures were as follows: The profile with one layer struts was excavated to 8.5m deep with the cantilever piles, then the struts were installed and the pre-stress of about 400~500kN was applied to the struts. As to the profiles with two layers of struts, it was firstly excavated to 5m deep, and the first layer of struts were installed at the position of 1.5m deep, then the pre-stress was applied; The next step was to excavate to 10m deep and install the second layer of struts at the position of 6.6m deep; Finally the pre-stress was applied to the second layer of struts and excavated to the bottom of the excavation.

The complete and reasonable results were obtained due to the protection of the instruments.

4. Monitoring results and basic action of the retaining structure

1) The force of the struts: The allowable force of the struts is about 3000kN, and the measured force of the struts is 1000~2200kN, therefore the struts are safe.
2) The lateral displacement of the piles: The maximum lateral displacements of the piles are 9mm~33.5mm and the average results are 23mm. The pile deflection curve at the opposite side of the strut is not symmetrical, so the retaining structure designer must pay special attention to the phenomenon. The maximum lateral displacement of the piles at the one layer struts profile occurred at the top of the pile, but the maximum lateral displacement of the piles at the two layers struts profile occurred in the middle of the pile.
3) The stress of the pile reinforced bars: The monitoring results of the reinforced bar stress were about 70~80MPa, only one-fourth of the steel strength.
4) The ground water level: The ground water level fell slowly down about 0.4~2.0m during the construction and rose slowly after construction, and finally it resumed to normal water level 9 months later.

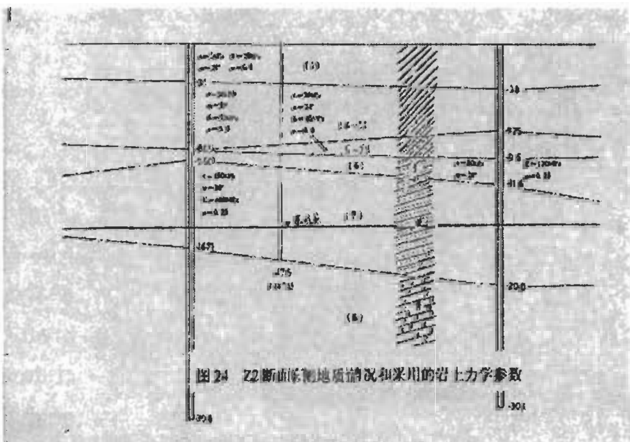

Fig.5 Geological profile at Z2

5. Analysis of the results

The monitoring of the project focused on profile Z2 and Z5. Profile Z2 had one layer of struts and profile Z5 had two layers of struts. A comparison between the theoretical and monitoring results was presented as the following, which was important for improving the design method. The analysis method used in the analysis was the simplified method for the piles under lateral loads proposed by Prof. P. Y. Lu.

1) Profile Z2

The geological profile is shown in fig.5. The monitoring results of ground water level and struts stiffness were used in the analysis of profile Z2. The pile deflection comparison between the results of the monitoring and the analysis before the struts installed are shown in fig.6. The comparison after excavated to the bottom is presented as fig.7.

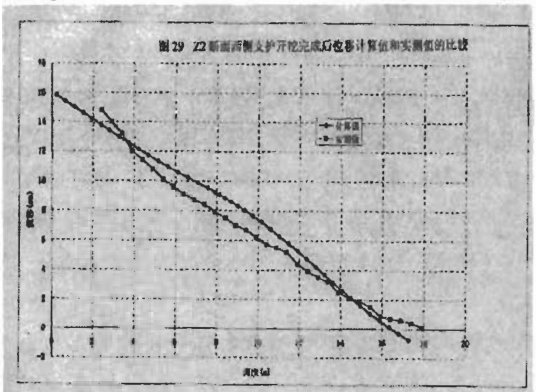

Fig.6 Pile deflection before the struts installed

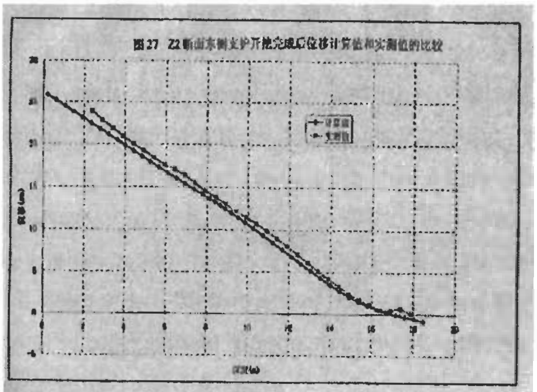

Fig.7 Pile deflection after excavated to bottom

Fig.6 and fig.7 proved that the analysis results of the pile deflection are consistent to those of the monitoring, however the analysis results of the pile moment defers from the monitoring results (see fig.8). There may be several reasons for it; such as too few monitoring points, and the affection of the tensile strength of concrete.

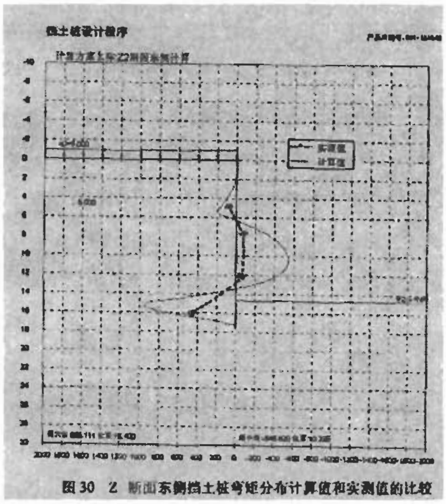

Fig.8 Pile Moment Comparison

2) Profile Z5

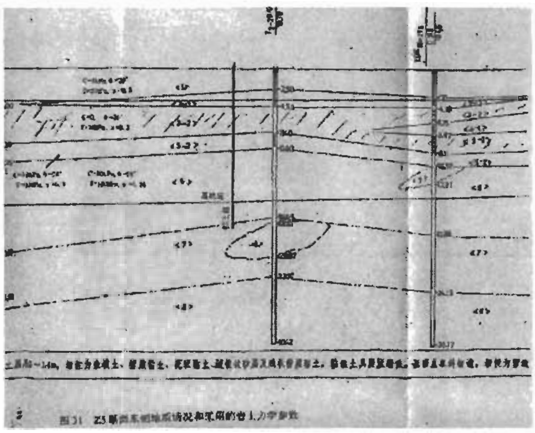

Fig.9 Geological profile of section Z5

The geological profile is shown as fig.9 and the analysis for profile Z5 was the same as that of profile Z2. Fig. 10 shows the pile deflection comparison between the monitoring and the analysis before the struts installed, and comparison after excavated to the bottom is given in fig.11. The analysis results of the pile deflection at profile Z5 are also consistent to those of the monitoring. The analysis results of the pile defer from those of the monitoring (see fig. 12) too.

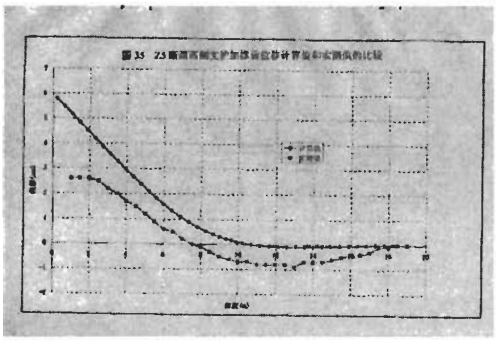

Fig.10 Pile deflection before the struts installed

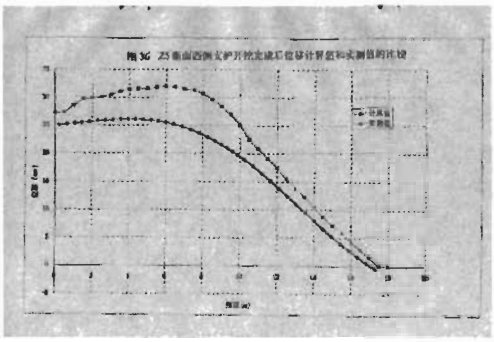

Fig.11 Pile deflection after excavated to bottom

6. Conclusions

The monitoring to the retaining structure of deep excavation of the Guangzhou Sport Center Subway Station was successful by obtaining the actual reaction of the stress and deformation of the retaining structure during the construction. The monitoring results prove that the pile deflection was consistent to those of the design, however there still existed a little difference for the strut force and pile moment to the previous estimation.

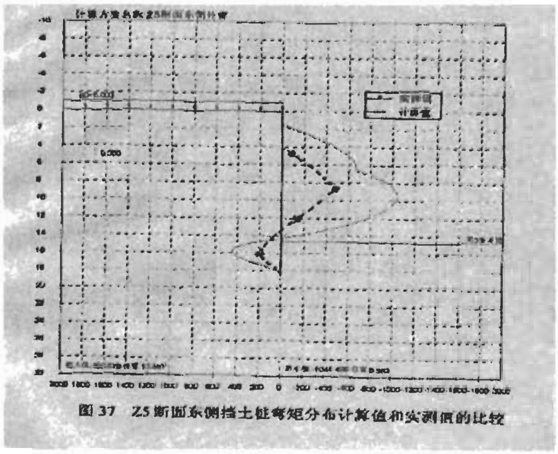

Fig.12 Pile moment comparison

鐵路土質邊坡滑坍搶修技術的研究

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Study of Rush-Repairing Technology for Soil Slope Landslide and Collapse on Railway Line

QIAN Chunyang and LIN Junxian
Science & Technology Institute, Guangzhou Railway (Group) Corporation
XIE Renqiu and XIE Qixin
Track Maintenance Department, Guangzhou Railway (Group) Corporation

撮要

論文通過對鐵路土質邊坡上不同土質、不同地形、不同入土深度、不同材質（鋼或木）的 128 根打入樁的水平靜載試驗研究，得到不同條件下，各種長度鋼、木抗滑樁所能承受的水平推力，爲土質邊坡滑坍搶修時如何選擇適當長度、排數的鋼樁或木樁提供參考。此外，根據研究和試驗成果，研究開發了《鐵路土質邊坡滑坍搶修設計軟體》，供搶修技術人員迅速確定最佳搶修方案。

Study of Rush-Repairing Technology for Soil Slope Landslide and Collapse on Railway Line

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[Abstract] In this paper, based on tests and research of transverse static load of 128 driven piles of different materials (steel or wood) driven in various soil and landform with different driving depth, the transverse capacity of steel and wood anti-slide piles with different length and conditions is obtained. It can be provided as reference for selecting proper length and number of rows of steel and/or wood piles in rush-repairing of soil slope landslide on railway line. In addition, a design software for rush repairing of soil slope landslide on railway line is developed on the basis of research and experimental results that can be used for rush-repairing technical personnel to decide optimum rush-repairing plan quickly.

1 Foreword

Landslide and collapse of subgrade of railway and highway, cut slope, and soil slope of other buildings happen from time to time due to flooding damage and other causes. Due to the lack of technical information of rush-repairing methods in these days, rush-repairing plan is decided mostly on experience. It is difficult to determine the optimal plan on site, which causes long discuss. As a result, rush-repairing time is delayed that brings about unnecessary loss. The purpose of this paper is to change this kind of situation and helps rush-repairing personnel to determine optimal rush-repairing plan as soon as possible to shorten rush-repairing duration. As the current rush-repairing methods include mainly driving in anti-slide piles, anti-press load in front of the slope, and cutting off soil load on top of the slope, the major researches in this paper is to obtain the ultimate transverse bearing capacity of anti-slide piles based on transverse static load tests of 128 driving-type rush-repairing anti-slide piles of different materials with various length that are driven in various soil and landform, so that rush-repairing personnel can scientifically determine the number and the length of piles for rush-repairing. At the same time, a software "Design and Calculation for Rush-Repairing of Soil Slope Landslide" has been developed based on the test results. By applying this software, rush-repairing technical personnel can swiftly determine the best rush-repairing plan, shorten rush-repairing duration, and reduce rush-repairing cost.

2 Experimental study on transverse static load of anti-slide piles for rush-repairing of soil slope landslide

2.1 Purpose, methods and reference standards for the test

The transverse static load tests of anti-slide piles for rush-repairing have been carried out in accordance with test methods close to actual operating conditions of rush-repairing. The test methods and procedure refers to relevant stipulations in P.R.C. industry standard JGJ94-94 "Constructive Pile Foundation Code", and the test loading is divided into 10 levels. In consideration of features of anti-slide piles for rush-repairing, the test terminating condition is determined as: pile body is broken off (for wood piles) or non-convergent displacement (for wood and steel piles). Take the level of load that is just before the pile is broken off or the displacement of the pile is not convergent as the ultimate transverse bearing capacity.

2.2 General situation of pile testing

The experimental site is at subgrade, cut, and flat ground the base of subgrade slope of operating railway line that belongs to Hunan Chenzhou Railway Section of Beijing-Guangzhou Line. The selected soil includes four types, namely, hard plasticity silty clay($I_L \leq 0.25$), plastic silty clay($0.25 < I_L \leq 0.75$), soft soil($0.75 < I_L \leq 1.00$), and swelling soil. The landform is divided into three types: flat base of slope, cut slope, and embankment slope. And soil type is divided into two kinds: undisturbed soil (base of slope or cut) and filled soil (embankment). The pile type includes driving wood pile and rail pile. For each test condition, three piles are tested (two or four piles for some conditions). The soil and landform of the site, pile' sizes, and buried depth by driving are listed in Table 2.3-1.

2.3 Test results and analyses

The results of transverse static load test of 128 anti-slide piles for rush-repairing are collected in Table 2.3-1.

Table 2.3-1

Pile type	Size (diameter m)	Soil type	Undisturbed soil or filled soil	Landform	Average ultimate transverse bearing capacity of anti-slide piled with various buried depth (kN)			Ultimate bearing capacity of pile with 1 m buried depth (kN/m)
					2.0m	3.0m	4.0m	
Wood pile	0.20	Hard plasticity silty clay	Undisturbed soil	flat slope-base	37.23	43.58	53.34	14.91
			undisturbed soil	cut slope	27.50	36.28	39.03	11.42
		Plastic silty clay	undisturbed soil	flat slope-base	22.50	30.00	45.00	10.83
			Fill	Embankment slope	17.50	27.50	31.67	8.52
		Soft soil	Undisturbed soil	flat slope-base	17.50	25.00	30.00	8.05
		Swelling soil	Undisturbed soil	flat slope-base	48.47			24.24
	0.12	Swelling soil	Undisturbed soil	flat slope-base		22.41		7.47
	0.20	Swelling soil	Undisturbed soil	cut slope	26.67	33.33	60.00	13.33
		Swelling soil	Fill	Embankment slope		55.00	57.50	16.07
Steel pile	50kg/m rail	Hard plasticity silty clay	Undisturbed soil	flat slope-base	47.43	48.97	82.50	19.88
		Hard plasticity silty clay	Undisturbed soil	cut slope	46.67	96.67	106.67	27.78
		Plastic silty clay	Undisturbed soil	flat slope-base	23.33	25.00	36.67	9.44
		Plastic silty clay	Fill	Embankment slope	17.50	40.00	47.50	11.67
		Swelling soil	Undisturbed soil	flat slope-base	46.67	91.02	173.33	34.58
		Swelling soil	Undisturbed soil	cut slope	40.00	46.67	96.67	20.37
		Swelling soil	Fill	Embankment slope	41.32	100.00	103.00	27.15

Table 2.3-1 shows that the factor of landform has no obvious influence on bearing capacity with weak regularity. If it is classified only according to the pile type, geology, and ultimate bearing capacity of the pile with buried depth of one meter, Table 2.3- 2 can be obtained. If it is classified according to the pile type, geology, and burying depth and average ultimate transverse bearing capacity of the pile, Table 2.3-3 can be obtained. And based on the data in table 2.3-2, Fig 2.3 (1) to (6): Correlation graphs between pile buried depth and its ultimate bearing capacity can be achieved.

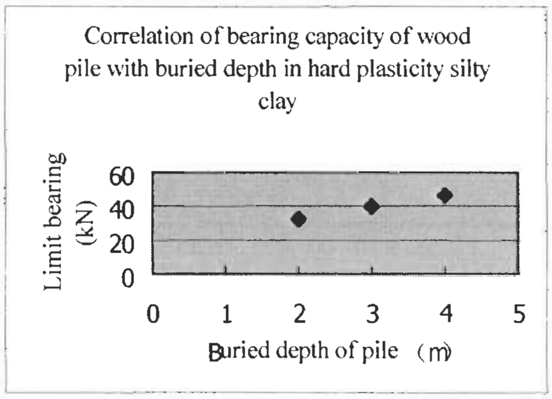
Table 2.3-2

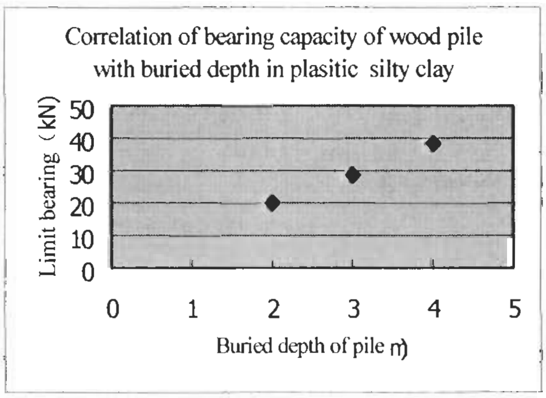
Pile type	**Size (diameter m)**	**Soil conditions**	**Ultimate bearing capacity of pile with 1 m buried depth (kN/m)**
Wood pile	0.20	Hard plasticity silty clay	13.17
		Plastic silty clay	9.68
		Soft soil	8.05
		Swelling soil	17.88
	0.12	Swelling soil	7.47
Steel pile	50kg/m rail	Hard plasticity silty clay	23.83
		Plastic silty clay	10.56
		Soft soil	27.37

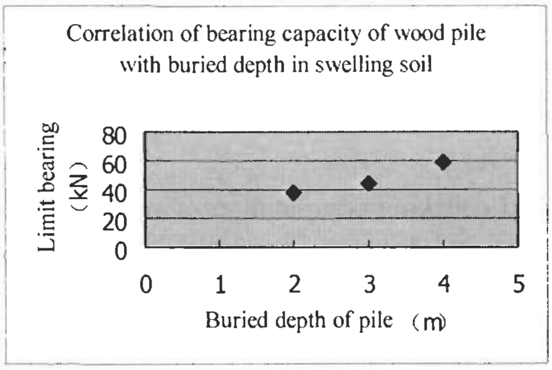
Table 2.3-3

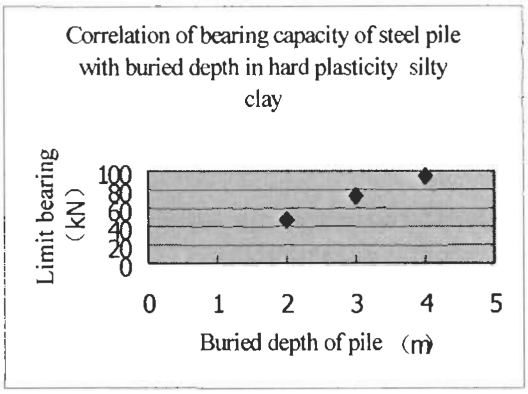
Pile type	**Size (diameter m)**	**Soil conditions**	Average ultimate transverse bearing capacity of anti-slide piled with various buried depth (kN)		
			2.0m	**3.0 m**	**4.0 m**
Wood Pile	0.20	Hard plasticity silty clay	32.37	39.78	46.19
		Plastic silty clay	20.00	28.75	38.34
		Soft soil	17.50	25.00	30.00
		Swelling soil	37.57	44.00	58.75
Steel pile	50kg/m Steel rail	Hard plasticity silty clay	47.05	72.82	94.59
		Plastic silty clay	20.40	32.50	42.09
		Soft soil	42.66	79.22	124.33

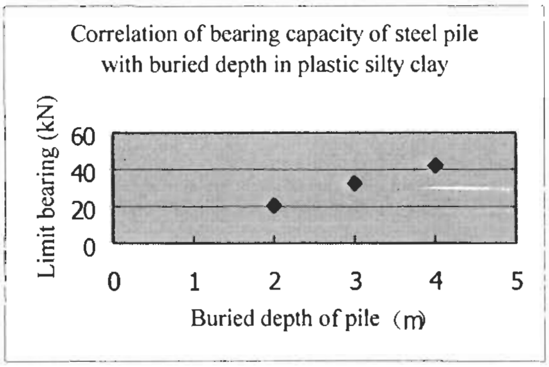
Fig 2.3 (1)-(6)

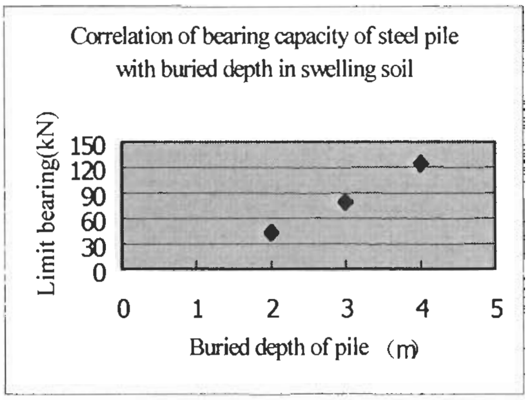












The correlation graphs of ultimate transverse bearing capacity with soil type and buried depth of different anti-slide pile types show that basically the transverse bearing capacity is proportionally linear with buried depth of the pile.

3 Research on software of design and calculation for rush-repairing of soil slope landslide

In order to aid technical staff of soil slope rush-repairing in quick determination of optimal rush-repairing plan, a set of "Design and calculation software for rush-repairing of soil slope landslide" has been developed by us cooperated with Beijing Lizheng Software Design & Research Institute, based on the data obtained from the transverse static load test of 128 anti-slide piles and the calculation methods recommended by current industrial code of PRC. Furthermore, based on calculation data by this software for landslide and collapse of soil scope on railway line. "Design Data Manual of Rush-Repairing for Soil Slope Landslide on Railway Line" has been worked out, which can be used as reference by rush-repairing personnel on site who do not familiar with computers.

3.1 Function of the software

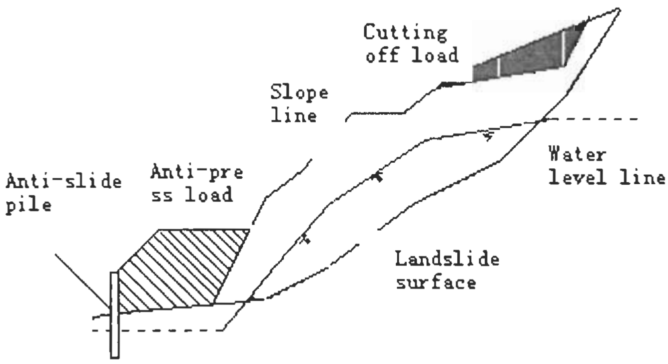
The software can run for automatic design and checking computation of anti-slide piles, anti-press load in front of the slope, and cutting off soil load on top of the slope for slope landslide correction. The three correction methods can be combined at will for use. And engineering workload and cost can be known quickly.

The main functions are listed as follows:

Objective of calculation		Description of functions
Automatic design	Automatic design of anti-slide piles	1. User selects several possible pile types from test database. 2. The software calculates thrust force of the landslide. 3. According to pile types selected by the user, automatically design and calculate the desire spacing and number of rows for each pile type. 4. Calculate the cost and construction period for each pile type that can be used by users to compare and choose.
	Automatic design of anti-press load in base of the slope	1. Automatically designs the start position of anti-press load, and calculates workload and cost. 2. Automatically design the height of anti-press load, and calculate workload and cost. 3. Plot: Variation correlation curve of height of anti-press load with thrust of landslide Variation correlation curve of height of anti-press load with volume Variation correlation of X coordinate position of anti-press load with thrust of landslide Variation correlation of X coordinate position of anti-press load with volume 4. Choose for users the most economic plan of anti-press load in base of the slope.
	Automatic design of cutting off load on top of the slope.	1. Automatically designs the start position of cutting off load on top of the slope, and calculates the workload and cost. 2. Plot: Variation correlation curve of Y coordinate position of top cutting off with thrust of landslide Variation correlation curve of Y coordinate position of top cutting off with volume 3. Choose for users the most economic plan of cutting off load on top of the slope.
Calculation checking	Calculation of safety coefficient of anti-slide piles	1. The user gives the pile types and distributions. 2. The software automatically calculates thrust force of the landslide. 3. Calculate safety coefficient of anti-slide piles. 4. Calculate the workload and the cost.
	Calculation of the thrust of the slope	1. The user provides the data of landslide. 2. The software automatically calculates thrust force of the landslide. 3. Calculate the workload and the cost.
	Calculation of the internal force of anti-slide pile	1. The user gives a pile type. 2. The software automatically calculates the thrust force of the landslide. 3. Calculate variation correlation of internal force of the pile, displacement, and soil pressure.
	Given C to back calculate ϕ	1. The user provides data of landslide. 2. The user gives cohesion on sliding surface. 3. Back calculate the angle of internal friction on the sliding surface according to the condition that the downslide force is equal to zero.
	Given ϕ to back calculate C	1. The user provides data of landslide. 2. The user gives the angle of internal friction on sliding surface. 3. Back calculate the cohesion on the sliding surface according to the condition that the downslide force is equal to zero.

3.2 Theoretic basis for calculation in the software

(1) The calculation graph for the rush-repairing design of slope landslide is shown in the following figure:



(2) Calculation of residual thrust of landslide mass:

For calculations of residual thrust of landslide mass of soil slope, the calculation methods described in "Subgrade" that is the design guidebook of railway engineering are utilized. The design formula is:

$$E_i = KW_i \sin\alpha_i + E_{i-1}[\cos(\alpha_{i-1} - \alpha_i) - \sin(\alpha_{i-1} - \alpha_i)tg\phi_i] - W_i \cos\alpha_i tg\phi - c_i l_i$$

$$= E_1 + E_2 + E_3 + E_4 \qquad (3\text{--}1)$$

where

E_i ---- The residual downslide force of Strip I of slide mass (kN/m);

The direction points to downslide direction.

E_i <= 0, no sliding occurs at given safety coefficient;

E_i > 0, sliding will occur at given safety coefficient.

The direction is in parallel with the sliding surface of Strip i;

E_{i-1} ---- The residual downslide force of Strip i-1 (kN/m);

The direction is in parallel with sliding surface of Strip i-1;

K ---- Considered desired safety coefficient for anti-slide calculation;

W_i ---- The weight of Strip i of landslide mass (kN/m);

α_i ---- The tilt angle of the sliding surface of Strip i (degree);

α_{i-1} ---- The tilt angle of the sliding surface of Strip i-1 (degree);

ϕ_i ---- The friction angle of the sliding surface in Strip i (degree);

c_i ---- The unit cohesion of the sliding surface in Strip i (kPa);

l_i ---- The length of the sliding surface in Strip i (m);

E_1、E_2、E_3、E_4 ---- The downslide force generated by this strip of slide mass, the downslide force and anti-slide force generated by strip i-1 of slide mass, anti-slide force generated by this strip of slide mass, and anti-slide cohesion generated by sliding surface of this strip of slide mass, respectively.

After the residual downslide force is calculated, it will be compared with ultimate bearing capacity of typical anti-slide piles, so that the required quantity, length, and number of rows of anti-piles can be figure out directly. If anti-press load is adopted in base of the slope, the anti-press load is considered to be a part of landslide, which has effect of reducing the thrust of landslide. Its calculation methods are identical to ordinary landslide. For cutting off soil load on top of the slope, the soil load that is cut off will be deducted in volume calculation.

(3) The calculation of internal force and displacement of anti-slide piles

Besides designing the quantity and length of anti-slide piles, the software can calculate the internal force and displacement of anti-slide piles based on elastic foundation method, which can be used as reference for designers. Methods include the followings according to the adopted soil pressure coefficients in calculating the soil pressure under the landslide surface:

"m" method;
"c" method;
"k" method.

1) The calculation of soil pressure

$$p = k\Delta \quad (3\text{--}2)$$

$$k = ah^n \quad (3\text{--}3)$$

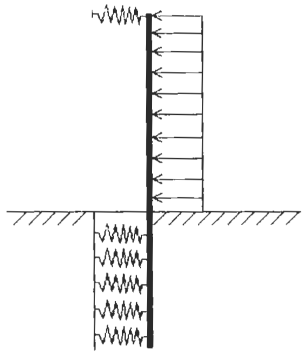
where

p ---- elastic soil pressure of piles under surface of landslide (kPa);
k ---- elastic soil pressure coefficient;
Δ ---- displacement of piles under surface of landslide (m);
a、n ---- calculation coefficients;
h ---- vertical distance from arbitrary point under surface of slide to the surface of landslide (m);

With the change of calculation coefficients a and n, there are different calculation methods.

If $n = 1$, $a = m$, it is named as "m" method;
If $n = 0.5$, $a = c$, it is "c" method;
If $n = 0$, $a = K$, it is "k" method.

2) Finite element equation



计算模型
Calculation model

$$\left[[K_z]+[K_T]+[K_{T0}]\right]\{\delta\}=\{p\}$$

(3--4)

where

$[K_z]$ ---- elastic rigidity matrix of anti-slide piles;

$[K_T]$ ---- elastic rigidity matrix of mass under landslide surface;

$[K_{T0}]$ ---- initial elastic rigidity matrix of mass under landslide surface;

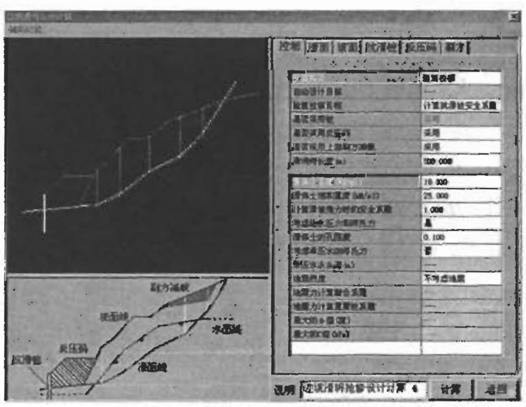
$\{\delta\}$ ---- displacement matrix of anti-slide piles;

$\{p\}$ ---- load matrix of anti-slide piles.

By substituting displacement boundary conditions into the above-mentioned equation and solving the equation, the displacement and internal force at all points of the pile can be obtained.

The boundary conditions of the base point of the pile are: free, simple supporting, and building in.

3.3 Typical interface of software



3.4 Engineering calculation example (omitted due to the limitation of paper's length)

4 Conclusion

(1) This paper describes the experimental research on transverse static load of 128 rush-repairing anti-slide piles of different types with various depth driven in different soil and landform conditions, which can provide relatively high reliable reference data for transverse ultimate bearing capacity of anti-slide piles of rush-repairing for soil slope landslide.

(2) The test results indicate that the ultimate transverse bearing capacity of anti-slide piles on railway embankment and cut slope is not reduced regularly in comparison with that on flat base of slope, and there is no much difference in their average values.

(3) The test results indicate that for uniform soil layer, the ultimate transverse bearing capacity of anti-slide piles of different type driven in various soil is proportionally linear with the buried depth of piles.

(4) "Design software for rush-repairing of soil slope landslide" is developed on the basis of experimental data and current theory, which provides technical personnel of slide rush-repairing with relatively accurate, rational and quick way for design and calculation.

(5) The achievement in this paper can be used not only for design of rush-repairing for railway soil slope, but also for rush-repairing for slope landslide on highway or other buildings.

References

[1] Science and Technology Institute, Guangzhou Railway Group Corporation, "Test Report of Transverse Static Load of Anti-slide Piles of Rush-repairing for Soil Slope Landslide", October 2001.

[2] Science and Technology Institute of Guangzhou Railway Group Corporation, Track Maintenance Department of Guangzhou Railway Group Corporation, and Beijing Lizheng Software Design and Research Institute, "Programming Principle and Operating Instructions for Design Software for Rush-Repairing of Soil Slope Landslide on Railway Line", December 2000.

[3] The First Survey and Design Institute, Ministry of Railways, the design guidebook of railway engineering: "Subgrade", *China Railway Press*, the third printing, 1995.

[4] WANG Xiaowen and QIAN Chunyang, "The Finite Element Analysis for Deformation and Internal Force of Shoring Structure of Deep Foundation Pit", *Railway*

Journal, 6 (1999).
[5] QIAN Chunyang, "The Soil Engineering Parametric Statistical Analysis and Study for Foundation Pit Calculation", *Guangdong Civil Construction*, Special issue for test technology, 2000.

九廣鐵路馬鞍山支線的岩土技術

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GEOTECHNICS OF THE KCRC MA ON SHAN RAIL

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撮要

馬鞍山鐵路 (MOS RAIL) 是九廣鐵路公司 (KCRC) 東鐵支線工程 (East Rail Extensions) 中三條新支線之一。 本鐵路共分四個土木工程合約。本文作者論述有關馬鞍山鐵路岩土技術的三個主要事項，分別爲：(a) 工程地質 (b) 樁柱地基 (c) 岩土儀器應用於監察與保護現有結構和鐵路。

關於馬鞍山鐵路的其他工程細節，作者已另闢專題文章在本會議中同時發表，敬希垂注。

GEOTECHNICS OF THE KCRC MA ON SHAN RAIL

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ABSTRACT: The Ma On Shan (MOS) Rail is one of the three extensions in the Kowloon-Canton Railway Corporation (KCRC) East Rail Extensions (ERE) project. It comprises four civil construction contracts. In this paper, the authors discuss three major geotechnical aspects of the MOS Rail including (a) engineering geology (b) piled foundations and (c) use of geotechnical instrumentation for protection of existing buildings and railway structures. More details of each construction contract can be found separately in the companion papers which have also been submitted to this conference.

INTRODUCTION

The MOS Rail provides a domestic passenger service from the existing KCRC East Rail station at Tai Wai to Wu Kai Sha, see Figure 1. The 11.5 kilometre railway is built primarily on a viaduct with a small section in the middle running at grade along the central median of the Tate's Cairn Highway and will have a maintenance centre at Tai Wai together with nine stations along the route. The key statistics of the MOS Rail are shown in Table 1. The construction work commenced in early 2001 and is expected to complete by end of 2004.

TABLE 1: KEY STATISTICS OF THE MOS RAIL

Alignment length	11.5 km
At grade section	2.3 km
Viaduct section	9.2 km
Journey time	15 minutes (from Wu Kai Sha Station to Tai Wai Station)
Number of stations	9 stations: Tai Wai, Che Kung Temple, Sha Tin Wai, City One, Shek Mun, Tai Shui Hang, Heng On, Ma On Shan and Wu Kai Sha.
Train size	4 cars
Maximum passengers per car	335 passengers
Civil construction contracts	4 civil construction contracts:- • TCC200 - Tai Wai to Shek Mun • TCC300 - Shek Mun to Wu Kai Sha • TCC400 - Tai Wai Station • TCC500 - Tai Wai Maintenance Centre

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[2] Senior Engineer, Binnie Black & Veatch Hong Kong Limited.
[3] Senior Engineer, Halcrow China Limited.
[4] Associate, Maunsell Consultants Asia Limited.
[5] Associate, Ove Arup & Partners Hong Kong Limited.

ENGINEERING GEOLOGY

The geology in the Sha Tin and Ma On Shan areas can be described as a sequence of faulted, folded and mildly metamorphosed volcaniclastic rocks that are extensively intruded by younger igneous rocks and partially overlain by a variety of superficial deposits (GCO,1987).

Ground Conditions

The stratigraphy revealed by the ground investigations is shown in Table 2. The alignment of the MOS Rail situates outside the Government's Schedule Area No. 4, as defined in PNAP 161 (Buildings Department, 1993), marble and other solution features have not been found during the ground investigations or construction.

TABLE 2: SUMMARY OF STRATIGRAPHY ALONG MOS RAIL

Stratum	Thickness (m)	Description
Fill	1 to 15	Highly variable, comprises (a) soft to firm sandy CLAY to sandy SILT or (b) loose to medium dense silty SAND, with occasional sub-angular size to cobble size rock fragments.
Marine Deposits	0 to 25	(a) Soft to firm, dark greenish grey silty CLAY (Marine Clay) with occasional shell fragments. (b) Loose to medium dense, slightly clayey silty SAND (Marine Sand) with occasional shell fragments.
Alluvial Deposits	0 to 30	(a) Soft, dark grey, slightly sandy silty CLAY to firm, dark grey sandy clayey SILT (Alluvial Clay/Silt) with fine gravel. (b) Loose to very dense, grey, clayey, silty, fine to coarse SAND with some gravel (Alluvial Sand). (c) Medium dense to very dense coarse GRAVEL and COBBLE.
Saprolites (Grade IV & V weathered rock)	10 to 110	Highly decomposed (HDG) to completely decomposed granite (CDG) comprises medium dense to very dense brown to pinkish brown, slightly clayey, silty fine to coarse SAND with occasional fine to medium sub-angular gravel.
Bedrock (Grade II & III Rock)	0 to > 10	(a) Moderately strong to strong, pinkish-grey, white spotted, moderately to slightly decomposed, medium to coarse-grained GRANITE with closely to medium spaced joints. (b) Extremely weak to strong, grey spotted, dark green and pink, moderately to slightly decomposed, medium grained MONZONITE.

Geological sections along the MOS Rail alignment, from Tai Wai to Shek Mun and from Shek Mun to Wu Kai Sha are shown in Figures 2 and 3 respectively. The general ground level ranges from approximately +6mPD at Tai Wai to approximately +22mPD at Wu Kai Sha. The solid geology is dominated by coarse-grained Granite, with local presence of Monzonite. Piezometers installed during the site investigation indicated that the groundwater was generally hydrostatic and that the levels ranged from the highest +8mPD to the lowest +1.1mPD.

Engineering Soil and Rock Properties

A summary of the engineering properties of the geological materials is given in Table 3.

TABLE 3: GEOTECHNICAL PROPERTIES OF SOILS & ROCK

Stratum	γ Mg/m^3	SPT N	PI %	c_u kN/m^2	c' kN/m^2	ϕ'	k m/s	E' MPa
Fill	1.6 to 2	5 to 40	18 to 27	23.5 to 188	0	30° to 37°	1E-6 to 2E-4	7 to 12
Marine Deposits (clay/silt)	1.25 to 1.8	3 to 12	22	15 to 60	5	28°	1E-10 to 1E-7	7
Marine Deposits (sand)	1.5 to 2	2 to 20	13	N.A.	0	33° to 35°	1E-6	8
Alluvial Deposits (clay/silt)	1.5 to 2	5 to 30	25	25 to 150	5	28° to 35°	1E-10 to 5E-6	4 to 13
Alluvial Deposits (sand)	1.7 to 2	4 to 50	15	N.A.	5	33° to 35°	1E-6 to 5E-6	13 to 25
Saprolites (Grade V weathered rock)	1.7 to 2	10 to 125	20 to 27	N.A.	5	35° to 38°	1E-6 to 1.6E-6	10 to 125
Bedrock (Grade II to III rock)	Uniaxial compressive strength (UCS) = 43 to 167 MPa Point load index = 0.5 to 7.5 MPa Young's modulus based on laboratory tests = 2 to 15 GPa Rock joint shear strength: c' = 0 to 100 kPa; ϕ' = 40°							

Geotechnical Hazards

The above ground conditions posed the following hazards and problems to the geotechnical works, in particular piling:-

Presence of 'rock cliffs' – The weathering zones vary substantially in thickness over short horizontal distances, resulting in large variation in rockhead elevation, even within the same pile group. This phenomenon together with the so-called '45-degree influence line' rule (i.e. a pile should not be intercepted by lines radiating at an angle of 45° from the toe levels of adjacent piles) could lead to very deep pile founding levels in rock.

Microfracturing of rock – The geological structure of the area is dominated by the southwest-northeast trending Lai Chi Kok - Tolo Channel Fault. A considerable number of minor vertical and sub-vertical faults are also present. Faulting presents a potential hazard to piling work, where extensive rock fracturing/crushing, hydrothermal alteration or increased depth of weathering has occurred. The ground investigations and the pre-drilling works revealed significant microfracturing of the granite bedrock in some areas, leading to reduced unconfined compressive strength (UCS) (10 MPa or less). Piles in these areas needed to penetrate deeper in order to attain Grade III or better rock with the UCS of 25 MPa or better.

Presence of corestones – Corestones (Grade IV/III/II) in saprolites were observed to vary between 0.5m to 3.5m in thickness. Apart from making the identification of the bedrock difficult, these boulder-sized corestones also formed obstructions to the piling works.

Presence of soft compressible deposits – The alignment is underlain by soft compressible deposits including Marine Deposits and Alluvial Deposits up to 25m thick. Piling in these soft deposits could lead to 'overbreak' more than 100 %. The high percentage of overbreak occurred either as a result of pile excavation in soft soil (particular in areas where temporary casings were not used) or as the fresh concrete squeezed into the soft soil layer during extraction of temporary casings.

PILED FOUNDATIONS

Piling was the largest geotechnical element in the project. In total, some 2900 piles were constructed, including end-bearing bored piles with bell-outs or rock sockets, frictional bored piles and pre-bored H-piles. A summary of the major piled foundations in the MOS Rail is given in Table 4. The details can be found in Chan et al. (2002), Tyson et al. (2002) and Yin et al. (2002).

Choice of Pile Type

Prior to detailed design, a value engineering review for the choice of foundation type was conducted for each construction contract. Factors considered included the cost, ground conditions, maximum pile capacity, local experience, availability of plants and labour, production rate and the ease to meet the statutory requirements and to obtain the Buildings Department's (BD's) approval.

In areas between Tai Wai and Heng On and between Ma On Shan and Wu Kai Sha, where the rockhead levels are relatively shallow (Figures 2 & 3) (between 5m and 70m below the ground level), end-bearing pile in Grade II or Grade III bedrock is considered to be the most appropriate pile type. The use of enlarged pile bases (bell-outs) reduces the total pile lengths and had been adopted in the Engineer's conforming design in all four construction contracts. Some contractors, however, proposed equivalent straight-shafted rock sockets as alternative designs, so as to suit their piling plants. Between Heng On and Ma On Shan, where the rockhead is deeper than 150m below ground level (Figure 3), the use of frictional bored pile is more appropriate. Shaft friction is provided by the Alluvial Deposits and by the CDG. For the ancillary buildings inside the proposed Tai Wai Maintenance Centre, where pile loads are relatively light (ranged between 2000kN and 9500 kN), the value engineering review indicates that they are best supported by groups of pre-bored H-piles.

Consultation with the Buildings Department

The MOS Rail works, including piling, have been exempted from the BD's approval of plans and consent to commencement. Instead, a 'consultation' process is undertaken. 'Consultation' means the submission of drawings, plans, calculations and other details, for vetting and agreement by the BD or the various consultation committees prior to the operation of the railway. Like all other construction projects in Hong Kong, the statutory requirements on testing of foundation works in PNAP 66 (BD, 2000a) and quality supervision in PNAP 242 (BD, 2000b) have to be complied with by the MOS Rail project. All relevant foundation layout plans and design information are submitted after completion of foundation works, together with piling records and proof tests results. Also, site safety plans are submitted to the BD before commencement of relevant construction works. Effects on existing or proposed nearby non-KCRC buildings need to be assessed by the Engineer as well.

TABLE 4: DETAILS OF THE MAJOR PILE TYPES IN THE MOS RAIL PROJECT

Pile Type	Total No.	Diameter (mm)	Length (m)	Details	Proof tests or verification
End-bearing bored piles with bell-outs	465	Shaft 1350 to 2900 Bell-out 1840 to 4350	9 to 72	Construction method • soil excavation by mechanical grab with full casing down to the rock head • bell-outs in rock formed by RCD machines Design parameters and criteria • ratio of bell-out diameter to shaft diameter = 1.5 (max.) • allowable end-bearing capacity in rock = 5MPa (Grade III) or 7.5MPa (Grade II)	• pre-drilling for 100% piles • Koden tests for 100% piles • sonic logging for 100% piles • interface coring for 100% piles • full length coring for 10 - 20% piles
End-bearing bored piles with rock sockets	1096	730 to 2500	7 to 89	Construction method • soil excavation by mechanical grab with full casing or drilling bucket with pile bores supported by drilling fluid • rock sockets formed by RCD machines or roller bit core barrels Design parameters and criteria • 1D < L < 1.5D where L = rock socket length, D = pile diameter • allowable end-bearing capacity in rock = 5MPa (Grade III) or 7.5MPa (Grade II) • allowable rock socket friction = 0.5MPa for both Grade II and III rock	• Same as end-bearing bored piles with bell-outs, except that Koden tests are not required. • For piles constructed with drilling fluid, post construction proof tests are required to demonstrate that the sockets are free of drilling mud.
Friction bored piles	207	2000 to 2500	30 to 88	Construction method • soil excavation by an auger and/or a drilling bucket, pile bores supported by drilling fluid Design parameters and criteria • allowable friction < 0.45 x SPT 'N' value, with a limit of 55 kPa • allowable end-bearing capacity < 5 x SPT 'N' value, with a limit of 1MPa	• pre-drilling for 100% piles • sonic logging for 100% piles • full length coring for 10 - 20% • 4 No. load tests on working piles; loaded to 2 times working load • post construction proof SPT for 5% of the number of piles
Pre-bored H-piles	1109	600	3 to 67	Construction method • piles formed by the "ODEX" system i.e. down-the-hole hammer with full length casing Design parameters and criteria • rock/grout bond strength for compression = 0.7 MPa (PNAP66) • grout/H-pile bond strength = 0.48 MPa (PNAP66) • shear studs were adopted to enhance gripping between steel and concrete • allowable end-bearing capacity in rock = 5 MPa in Grade III rock	• pre-drilling for each group of piles • post construction proof drilling for 1% of the number of piles • load tests for 1% of the number of piles; loaded to 2 times working load

Piling Specification

All four civil construction contracts in the MOS Rail share the same standard piling specification, as well as similar particular specification. A few special aspects of the MOS Rail piling specification are discussed below:-

Putting emphasis on proof tests and verification tests – The MOS piling specification has incorporated all the statutory requirements in the PNAP 66 regarding pre-drilling and post construction proof tests, see Table 4. In addition, it also requires all bored piles to be tested for their integrity using sonic logging method. For piles with enlarged bases, Koden tests are to be used to confirm the as-built dimensions. To mitigate against the uncertainties in the determination of founding level, in areas where the rockhead is sloping steeply, additional boreholes (extra over those required in PNAP 66) are required to be drilled by the Contractor around the pile circumference for further confirming the rockhead level.

Promotion of good workmanship – An example is that in the MOS Rail only reverse circulation drilling (RCD) machines or roller bit core barrels are allowed to form bell-outs or rock sockets. The use of chisel is prohibited. This is to eliminate the unnecessary vibrations caused to adjacent structures and to ensure the integrity of the rock sockets and bell-outs.

Encouragement of innovations from contractors – The piling specification permits contractors to introduce innovative measures for improving the quality of the works. An example of innovative idea from contractors is the introduction of cement grout slurry to the pile bore prior to concreting so as to minimise the risk of concrete segregation at the pile base.

PROTECTION OF EXISTING STRUCTURES

The MOS Rail was constructed in one of the most densely populated district in the New Territories. The Tai Wai Station is one of the busiest KCRC railway stations. To enable the construction of the new Maintenance Centre and the new Tai Wai Station, extensive modification works were carried out at the existing East Rail railway embankment and at the existing Tai Wai Station. These modification works included the construction of two subways under the existing railway embankment and installation of permanent retaining walls in the vicinity of the existing station and railway embankment. Details of these construction works have been described elsewhere (Tyson et al., 2002 and Yin et al., 2002).

Response Values and Response Actions

To protect the existing structures from damaging and to prevent disruption of the railway service, geotechnical instruments had been installed at strategic locations and were monitored during construction. Each instrument had been assigned three 'Response Values', as defined in the following sections.

Alert value – This is approximately 50% of the allowable structure or ground deformation. This is the lowest Response Value and signifies the reading for an instrument at which closer attention needs to be paid to the instrumentation data in case of subsequent reaching of an Action Value. Examples of response actions which need to be taken at this stage are: increase the frequency of monitoring or improvement of the construction method.

Action value – This is approximately 70% to 80% of the allowable structure or ground deformation. This is the intermediate Response Value and signifies the reading for an instrument at which response action is mandatory. Examples of response actions which need to be taken at this stage are: tamping of railway track ballast, provision of extra external support to retaining structures or carry out ground improvement such as grouting.

Alarm value – This is approximately 100% of the allowable structure or ground deformation. This is the highest Response Value and signifies the reading for an instrument which shall not be reached. Examples of response actions which need to be taken at this stage are: suspension of all activities, stabilization of the excavation face with shotcrete or backfill excavation with concrete.

Of particular interest is the definition of the maximum allowable deformations for the existing East Rail structures, which are largely based on the Works Branch Technical Circular 21/95 (Works Bureau, 1995) together with the in-house guidelines from KCRC, see Table 5.

TABLE 5: MAXIMUM ALLOWABLE DEFORMATIONS OF KCRC EAST RAIL STRUCTURES

Structures	Maximum Allowable Deformations	Instrumentation Adopted for Monitoring
Rail tracks	• Vertical movement – 20mm • Horizontal movement – 10mm • Angular distortion – 1 in 500 • Differential settlement between rails – 5mm • Change in cant over 2m of rail track – 6mm	• Automatic deformation monitoring system (ADMS) mounted on sleepers • Tiltmeters
Overhead Line mast	• Angular distortion – 0.54 degree • Vertical movement – 10mm	• Automatic deformation monitoring system (ADMS) mounted on sleepers • Tiltmeters • Deformation markers
Platform and station structures	• Total settlement = 25mm • Differential settlement = 1:1000	• Liquid level gauges • Deformation monitoring points
Vibrations	Vibration in the vicinity of railway structures – 15mm/s	• Seismographs

Geotechnical Instrumentation and Lines of Communication

The main instrumentation for monitoring the rail track is the Automatic Deformation Monitoring System (ADMS), which comprises optical reflective prisms installed on the sleepers. These prisms are read by electronic 'total stations', which have connected to computer terminals for processing the data. These prisms had been installed at 2-metre centres in areas where the construction works were intensive. These became wider spaced in areas where the works were less intensive. The ADMS has the advantage of being able to survey the deformation of the rail in three directions (vertical, transverse and longitudinal) within a short period of time. However, the ADMS can only measure movements of the sleepers. Therefore, the measurements need to be converted to the rail distortion based on correlation developed initially by manual surveys. Monitoring results are automatically emailed to all designated parties immediately after the deformations have been calculated. Appropriate response actions will then be taken. As a precautionary measure, all excavation works underneath the rail tracks have to be carried out in the early morning during "track possession time" (between 1:00am and 4:30am). Moreover, a Line Safe Message is delivered by the resident site staff to the East Rail Control Centre every morning by 5:00am before the start of traffic.

CONCLUSIONS

Much has been gained from the geotechnical works carried out for the MOS Rail. The ground investigation information supplemented the existing published geological information for the Sha Tin/Ma On Shan area. The various types of piled foundations set the precedence for future foundations construction in the nearby areas. The geotechnical instrumentation has successful monitored the behaviour of the existing railway structures during the construction of the MOS Rail and protected them from potential damages.

REFERENCES

Buildings Department (1993). *Practice Note for Authorized Persons and Registered Structural Engineers (PNAP) No.161 – Development in the Area Numbers 2 & 4 of Scheduled Areas.* Buildings Department, Hong Kong.

Buildings Department (1995). *Practice Note for Authorized Persons and Registered Structural Engineers (PNAP) No.141 – Foundation Design Building (Construction) Regulations 1990 – Part VI.* Buildings Department, Hong Kong.

Buildings Department (2000a). *Practice Note for Authorized Persons and Registered Structural Engineers (PNAP) No.66 – Pile Foundations.* Buildings Department, Hong Kong.

Buildings Department (2000b). *Practice Note for Authorized Persons and Registered Structural Engineers (PNAP) No.242 – Quality Supervision Requirements for Foundation Works.* Buildings Department, Hong Kong.

Chan, P. W. F., Chan, W. K. H., Tse, C. M. (2002). "Design and Construction of Bored Piles in Completely or Moderately Decomposed Rock for the KCRC Ma On Shan Rail." *Geotechnical Aspects of Mass Transportation Systems.* Proc., 22nd HKIE Geotechnical Division Annual Seminar, Hong Kong Institution of Engineers, Hong Kong. In preparation.

Geotechnical Control Office (1987). *Geotechnical Area Studies Programme Report II. Central New Territories.* Geotechnical Control Office, Hong Kong, pp 15-25.

Tyson, J. P., Harman, P., Swann, L., Leung, S. K., Tse, C. M. (2002). "Geotechnical Design of the Extension to Tai Wai Railway Station." *Geotechnical Aspects of Mass Transportation Systems.* Proc., 22nd HKIE Geotechnical Division Annual Seminar, Hong Kong Institution of Engineers, Hong Kong. In preparation.

Works Bureau (1995). *Capital Works or Maintenance Works in the Vicinity of the Kowloon-Canton Railway (Hong Kong) section. Works Branch Technical Circular No. 21/95.* Works Bureau, Hong Kong.

Yin, K. K., Kemp, S., Askew, I., Chor, C. K. C. (2002). "Geotechnical Aspects of the ERE Tai Wai Maintenance Centre." *Geotechnical Aspects of Mass Transportation Systems.* Proc., 22nd HKIE Geotechnical Division Annual Seminar, Hong Kong Institution of Engineers, Hong Kong. In preparation.

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九廣鐵路馬鞍山支線工程中之鑽孔灌柱樁的設計與施工

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Design and Construction of Bored Piles in Completely or Moderately Decomposed Rock for the KCRC Ma On Shan Rail

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撮要

爲應付當地居民的交通需求，九廣鐵路公司正在建造馬鞍山支線連接大圍至烏溪沙。該鐵路之全長將會爲11.4公里，其中8.4公里將會由高架旱橋承托，而高架旱橋的樁基礎將由建在全風化或中風化花崗岩地基上的約一千枝大直徑鑽孔灌柱樁承托。由於在恆安站至烏溪沙站一段的地下岩石太深，這一段的樁柱將會建在全風化石之中，樁柱的承托力來自柱身與泥土之摩擦力。其他的樁柱爲陷進岩石內的端承樁，其承托力來自樁端岩石的承載量和樁柱和岩石孔邊的摩擦力。本文詳述陷進岩石鑽孔灌柱樁與泥土摩擦力鑽孔灌柱樁的設計與施工，及施工時的接受測試，其中包括樁端連接面的鑽探測試，以及樁身的音波測試。工程中會有數枝泥土摩擦力鑽孔灌柱樁設有量度儀器，以作爲求證摩擦力設計的載重測試。本文亦詳述了其中的第一個樁柱靜壓力載重測試。

Design and Construction of Bored Piles in Completely or Moderately Decomposed Rock for the KCRC Ma On Shan Rail

by Paul W. F. Chan[1], Walter K. H. Chan[2] and C. M. Tse[3]

ABSTRACT: The KCRC is constructing the Ma On Shan Extension of the existing East Rail from Tai Wai to Wu Kai Sha to meet the transportation needs of the residents of the area. The total length of the rail will be 11.4 km, of which 8.4 km will be supported by viaducts. The foundations of these viaducts require the construction of approximately a thousand large-diameter bored piles in CDG to MDG. Some of the bored piles along the alignment from Heng On Station to Wu Kai Sha Station, where the rockhead is found at great depth, will be founded in CDG with their load carrying capacity derived from side resistance. The other piles are rock socketed tip resistance piles with their capacity derived from rock bearing strength and rock socket side resistance where necessary. The design and construction of rock socketed large-diameter bored piles and side resistance piles in CDG, and details of the acceptance tests including interface coring and sonic logging are presented in this paper. Several test piles would be constructed, instrumented and loaded to evaluate the adequacy of the design allowable side resistance for piles founded in CDG. Details of the first full-scale static compressive load tests are also presented in this paper.

INTRODUCTION

The Ma On Shan (MOS) Rail is an extension of the existing East Rail from Tai Wai to Ma On Shan to meet the transportation needs of the residents of the area. The project consists of two separate design packages, i.e., TDD-200 and TDD-300. The TDD-200 section traverses from Tai Wai to Shek Mun and comprises four passenger stations: Che Kung Temple Station (CKT), Sha Tin Wai Station (STW), City One Station (CIO), and Shek Mun Station (SHM), with a total length of 4 km of elevated tracks. The TDD-300 section is from Shek Mun to Ma On Shan and comprises four passenger stations: Tai Shui Hang Station (TSH), Heng On Station (HEO), Ma On Shan Station (MOS), and Wu Kai Sha Station (WKS). The total length of the TDD300 railway is 6.8 km, of which approximately 3 km will be constructed at-grade between SHM Station and TSH Station located in the median of Tate's Cairn Highway. The remaining section of the railway will be running on elevated viaduct structures. The viaduct piers and the station structures will be supported on pile foundations.

The design and construction of the different types of foundations to support the viaduct and station structures are presented in this paper. Since side resistance bored piles are used at locations where the bedrock is found at great depth, full-scale pile load tests are carried out as

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required by the Buildings Department (BD) to verify the pile design criteria. Details of the first pile load test and the test results are also presented in the paper. Details of pile construction including various pile integrity tests and some remedial works carried out to repair the defects found at the concrete/rock interface at pile base are also described.

GROUND GEOLOGY AND CHOICE OF FOUNDATION TYPES

Extensive ground investigations have been carried out during the design stage to reveal the subsurface conditions and to evaluate the soil and rock properties. In particular, the depth and properties of the bedrock are of particular interest for the design of the foundations of the viaduct piers and station structures.

The ground geology along the MOS Rail alignment can be described simply as a sequence of faulted, folded, mildly metamorphosed volcaniclastic rocks that are extensively intruded by younger igneous rocks, mostly granite, and partially overlain by a variety of superficial deposits. The alignment is outside Scheduled Area No. 4 and no marble bedrock is encountered in the boreholes. The superficial deposits are extensive and in places exceed 60 m in thickness. They comprise older alluvial gravel, sand and mud, covered by a layer of marine mud and/or sand. A mantle of completely weathered rock with thickness varying from about 10 m to more than 100 m in the Ma On Shan Area is found overlying the granite bedrock.

In view of the intense loading from viaduct piers and station structures, pile foundations are required to transfer the railway and superstructure loads to the load bearing strata deep below the ground surface. The use of driven steel H-piles have been considered during the preliminary design stage because of their relatively low cost. However, this option was rejected because of the adverse noise impact to the nearby residents during the pile driving process. This option is also not feasible at some pier locations where large piles caps necessitated by the use of large number of H-piles cannot be accommodated. Large diameter bored piles are thus considered appropriate and adopted for both viaducts and station structures.

As revealed by the drillholes sunk during the feasibility study and the design stage, Grade III or better granite bedrock are found at depths varying from a few metres to 60 m along the TDD200 alignment. For TDD300, the rockhead is found at shallow depth varying from 10 m to 50 m between the at-grade section and the middle of the alignment near HEO Station. At the northern end of the railway near WKS Station, the rockhead is also shallow at less than 20 m. Tip resistance bored piles are therefore considered suitable for the viaduct piers and seven out of the eight stations.

Along the remaining TDD300 alignment from north of HEO Station to WKS Station, the rock head is found at depth in excess of 100 m. At certain locations between HEO Station and MOS Station, bedrock is not found in some boreholes sunk to depths of more than 150 m. As the use of driven piles has been rejected, side resistance bored piles are considered suitable to support the viaduct structure and MOS Station. The side resistance bored piles are designed to carry the superstructure loads to load bearing soil strata of firm CDG material that would provide sufficient side resistance capacity without excessive deformation.

However, side resistance bored piles are not commonly used in Hong Kong and are not readily accepted by BD for use to support structures. Conceptual design proposal has been submitted to BD for consultation and their consensus on the design approach, methodology, design parameters and test requirements, etc. Full-scale loading tests have been agreed to be carried out to justify the pile capacities as well as the design parameters.

TIP RESISTANCE PILES

Using the presumptive allowable rock bearing pressures stipulated in PNAP141 (BD 1995), i.e., 5 MPa for pile founded on Grade III rock and 7.5 MPa for Grade II rock, piles with bell-outs at founding level are designed to spread the pile loading onto the bedrock to within the allowable bearing pressure. The bell-outs would have side walls inclined at 1 (horizontal) to 2 (vertical) and would be formed entirely in bedrock of the respective grade. Reverse circulation drilling (RCD) has been specified for excavation of the bell-outs in bedrock to maintain the rock integrity. Chiselling in bedrock would not be allowed. Koden tests would also be carried out to verify the dimensions of the bell-outs prior to concreting.

At the commencement of construction, the Contractor proposed to change the use of bell-outs at pile bases to rock sockets so as to suit their construction equipment. The additional bearing capacity provided by the bell-out would be compensated by the side resistance between the pile and rock along the socketed pile shaft. In their proposed design change, bored piles are designed to be socketed into Grade III or better rock with straight pile shafts. Both tip and side resistances are to be used to calculate the pile capacity. Assuming the concrete and rock behave as elastic isotropic continua and the bond along the rock-concrete interface is not broken, Pells and Turner (1979) have analysed the socket/rock system. Tomlinson (1994) outlined the study by Wyille (1991) on the detailed account of factors governing the load carrying capacity and settlement of rock socket and the study by Osterberg and Gill (1973) on the distribution of side wall shear stress in relation to socket length and modulus ratio. Tomlinson (1994) also presented the correlation of the unconfined compression strength of the rock and rock socket bond stress established by different researchers including Horvarth (1978), Rosenberg and Journeaux (1976) and Williams and Pells (1981). Despite the differences among different methods in the determination of the contribution of side resistance of rock socket, all the literature cited confirm that rock socket resistance can be taken into account in the determination of pile capacity.

In the design of the rock sockets, a unit side resistance of 500 kPa for socket in Grade III rock is adopted following the guideline given in GEO (1996). In accordance with PNAP 141, a permissible unit tip resistance of 5 MPa is adopted for Grade III or better rock. Furthermore, it has been agreed with BD after consultation that the length of pile socketed into bedrock providing side resistance, in addition to the tip resistance, should be within 1 to 1.5 times the pile diameter. BD imposed such a requirement in consideration of whether the side resistance and tip resistance can be mobilized simultaneously if the socket is either too deep or too shallow.

SIDE RESISTANCE PILES

The design of side resistance bored piles is based on the guideline given in GEO (1996). The piles shall be installed through the existing fill and superficial deposits, and embedded in adequate length in CDG to provide the design capacity.

Based on GEO (1996) Section 5.4.5(3) for large-diameter bored piles and barrettes installed in saprolite in Hong Kong, the SPT factor, i.e., the quotient of the average mobilized unit side resistance divided by the average SPT N-values, ranges between 0.8 and 1.4 (kPa). Adopting a safety factor of 2, the design unit side resistance should thus vary between 0.4N to

0.7N. As illustrated in the available drillhole logs along the alignment where side resistance piles would be used, the CDG are strong insitu soil of high SPT N-values of generally more than 30 underneath the superficial deposits, up to more than 100 at greater depth and close to 200 at the founding depth. Subsequent to our consultation with BD on the design principle as well as the construction method, the design criteria of the side resistance bored piles adopted are as follows:

- Design unit side resistance = 0.45×N with a maximum value of 55 kPa (N=122)
- Design unit tip resistance = 5×N with a maximum value of 1000 kPa (N=200)

In order to ensure that pile base loosening would not occur during pile construction, an additional requirement of N>100 at pile base has been adopted in the determination of pile founding level. Since weaker soils of lower SPT N-values would be more vulnerable to loosening during pile excavation, the additional requirement would ensure that the unit tip resistance of soil adopted in the design can be achieved.

When there are soft marine deposits (MD) in the superficial deposits, the MD layers would consolidate due to the loading of the existing reclamation fill as well as any additional loading atop. This would result in settlement of the MD layers and all the soil layers overlying them. These settling layers would induce a negative skin friction (NSF) on the piles. The NSF so induced is included in the pile loads in the design.

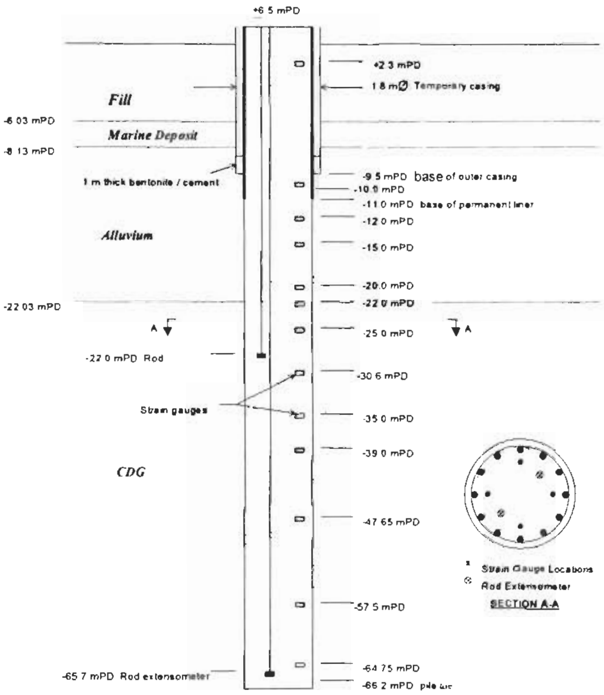
At the commencement of piling works, the Contractor proposed to use bentonite as the drilling fluid for the construction of bored piles between HEO Station and WKS Station as well as at MOS Station, including most of the side resistance piles. Temporary casing will not be lowered to the founding level. The pile bore below the bottom of the temporary casing at a few metres below ground surface will be supported by bentonite that will be displaced during concreting. The temporary top casing will also be withdrawn upon completion of concreting.

There is a general concern whether the bentonite construction method would affect the side resistance along the pile shaft. Based on the specialist subcontractor's experience and track records, the design unit side resistance value of 0.45N can be achieved using their proposed bentonite construction method. Side Resistance Piles No. 117A-5 south of MOS Station and No. 131-2 near Villa Athena were originally designed to be load tested to demonstrate the adequacy of pile capacities and the validity of the correlation between the unit side resistance and the SPT-N value adopted for design. Both piles were constructed using the bentonite method and load tested to twice the design working load. Moreover, BD required two more pile load tests when bentonite was used.

PILE LOAD TESTS

The first load test on a side resistance pile was carried out on Pile P117A-5, a 1.5-m diameter bored pile of approximately 72 m long founded in CDG. The test was commenced on 21 November 2001 and completed on 25 November 2001.

The Contractor proposed to install a sleeve along the pile length within and above the MD layers with an outer temporary steel casing of 1.8 m diameter and a permanent steel casing of 1.5 m. The bottom 1 m of the annulus between the two casings was filled with a weak bentonite cement grout. The pile bore was excavated under a bentonite slurry. Strain gauges were installed at 13 levels along the pile shaft from top to bottom and two extensometers were installed at the top level of the CDG layer and at the pile tip. Details of pile construction and instrumentation are shown in Fig. 1.



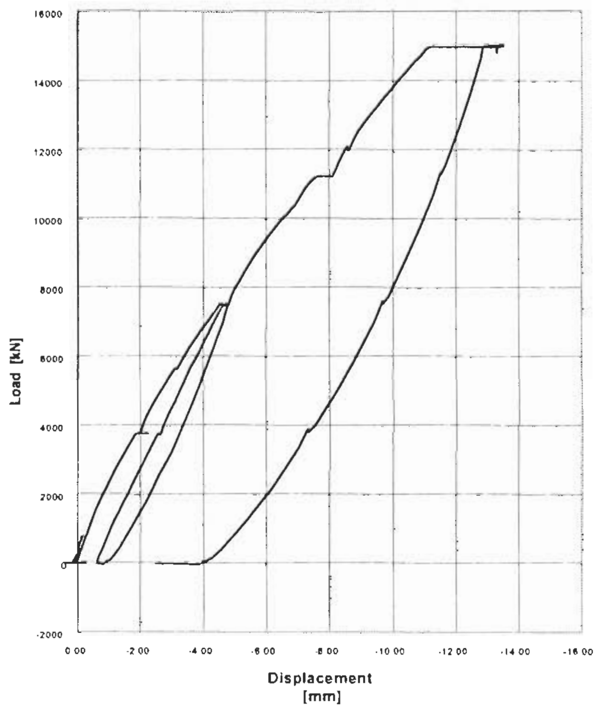


Fig. 1. Details of Test Pile Construction and Instrumentation

Fig. 2. Pile Head Displacement versus Test Load

The pile founding level and design pile capacity were determined on the basis of the pre-drill log sunk at the pile centre. Disregarding the contribution from the soil materials at and above the MD layers, which is separated from pile shaft by the sleeve, the design pile capacity is 7,500 kN with the pile founding level at –66.2 mPD. The pile was thus load tested to 15,000 kN.

The acceptance criteria of the loading tests stipulated in Appendix D of PNAP66 (BD 2000a) were adopted, i.e.,

(i) The maximum settlement at the pile head under the maximum test load of twice the working capacity should not exceed

$$\frac{2WL}{AE} + \frac{D}{120} + 4mm$$

where W = design working load (kN); L = length of the pile (mm); A = cross-section area of the pile (mm^2); E = Young's modulus of the pile material (kN/mm^2); and D = the least lateral dimension of the pile (mm).

(ii) The residual settlement at pile head after removal of maximum test load should not exceed D/120 + 4 mm.

The pile head settlement measured at the design working load is 4.8 mm and that at the maximum test load was approximately 12.8 mm. The residual settlement measured upon unloading was approximately 2.5 mm. A plot of the pile head settlement versus the applied test load is shown in Fig. 2. The allowable PNAP 66 settlements are 37.0 mm and 16.5 mm, respectively. Therefore, the PNAP 66 criteria are satisfied.

Moreover, two rod extensometers were also installed in the test pile at levels of –22 mPD and -65.7 mPD. Measurements made by the rod extensometers indicate a pile compression of 5.9 mm and 11.8 mm, respectively between the pile head and the two anchor levels, equivalent to a pile settlement of 6.9 mm and 1 mm at –22 mPD and –65.7 mPD (i.e. pile tip), respectively.

As shown in Fig. 1, there are 4 strain gauges installed at each of the 13 levels along the test pile to measure the pile strain induced by the test load and to evaluate the distribution of side resistance along the pile shaft. The strain gauges were automatically and continuously read by a computerized data logger after the pile was cast and throughout the test period. Because of the irrational readings obtained from a small number gauges, these gauges were considered to be malfunctioning after the concrete curing period. The readings from these few gauges were disregarded in the data analyses and graphical plots. The strain gauge readings obtained during the 1st and 2nd loading cycles, i.e., one and two times the design working load, are presented in Figs. 3 and 4, respectively. The distributions of pile strain along the pile shaft shown in the plots should reflect the load distribution along the piles.

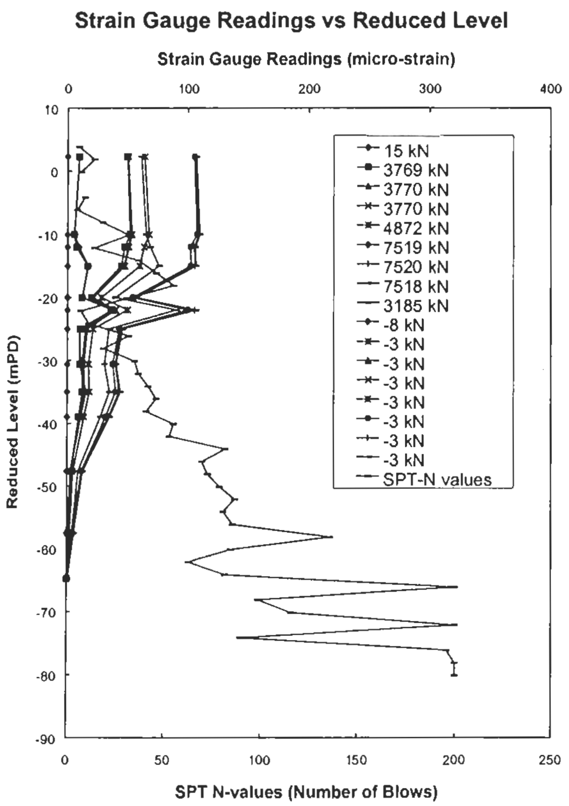


Fig. 3. Strain Distribution during 1st Loading Cycle

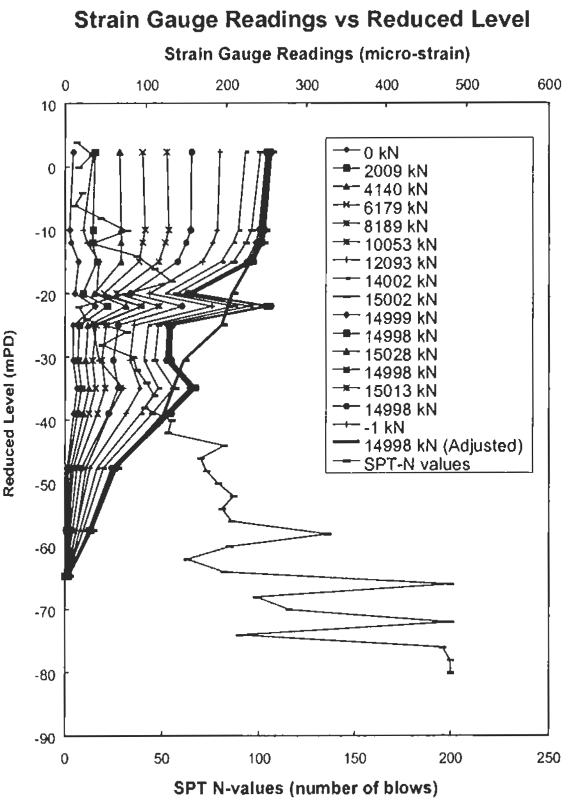


Fig. 4. Strain Distribution during 2nd Loading Cycle

Theoretically, the axial load in a pile should decrease with depth as the test load is applied at the top and the accumulative support provided by side resistance increases with depth. However, it can be observed in Figs. 3 and 4 that there are increases in pile load at two depth intervals, i.e., from –20 mPD to -22 mPD and from -30.6 mPD to –35 mPD. An obvious explanation of this anomaly is malfunctioning of the strain gauges at these two levels. However, the explanation cannot be substantiated because: (1) no anomalies were reported on the performance of the strain gauges at these two levels; and (2) the readings from all the four gauges at each of these two levels changed consistently during the test. On the other hand, it is worth noting that some irregularities were observed in the historical strain gauge readings logged since the construction of the test pile as provided by the specialist subcontractor. All the strain gauges recorded compressive strain prior to the load test. This is probably due to concrete shrinkage, which caused the steel reinforcement bars and the strain gauges under compression. However, the gauges at –22 mPD and –35 mPD recorded much lower values. Similar lower compressive strains were also observed in the gauges at the immediate lower level of –39 mPD. The phenomenon indicated by the gauges at these particular levels may be attributed to the localized concrete shrinkage cracking around the gauges. There may be restraints to concrete shrinkage provided by the surrounding soil that is a function of the shear strength of the soil. At locations of weaker soils, the localized restraints are smaller rendering easier cracking of concrete at these locations. It can be noted from Figs. 3 and 4 that the SPT N-values around these particular levels are relatively lower than those above and below. Therefore, the compressive strains are released around the cracks as indicated by the lower compressive strain gauge readings.

In view of the unexpected strain gauges readings at the levels of –22 mPD, –35 mPD and –39 mPD, some adjustments have been made to the readings from these and adjacent gauges under the maximum test load. The plot after the adjustment is also shown in Fig. 4.

Based on the adjusted strain gauge readings, the load carried by the piles at different lengths and the side resistance mobilized are calculated and tabulated in Table 1. The SPT load factor is also calculated by dividing the mobilized unit side resistance by the average N-values at that particular depth. The SPT load factor varies from 1.05 to 4.0 except at the lower depth where much lower values are obtained as the side resistance may not have been fully mobilized. These SPT load factor values may not be very accurate as they are obtained from strain gauges installed at several levels only, which may not accurately represent the behavior of all the soil surrounding the pile under test loads. In a report prepared by the Contractor's geotechnical consultant (GCG 2002), an approach of calculating *the average cumulative SPT load factor* by considering the average SPT values at and above the particular level against the summation of unit side resistance above that level was adopted. The average SPT load factor was suggested to be 2.1 from the pile head down to approximately –40 mPD.

Nevertheless, the test results indicate that, as shown in the plots in Fig. 4, the side resistance that can be mobilized would increase with the SPT N-values of the soil. The factor relating these two parameters, i.e., the SPT load factor, should be in the range of 1 to 2. As the pile head settlement under the test load of twice the design capacity was less than that allowed by PNAP 66 and the SPT load factor obtained in-situ is greater 2×0.45, the full-scale load test is considered to be successful in providing justifications for the design parameters and pile capacity.

Table 1: Summary of Results from Loading Test on Pile No. P117A-5

Strain Gauge Level (mPD)	-20.0	-22.0	-25.0	-30.6	-35.0	-39.0	-47.65	-57.5	-64.75
Strain Gauge Reading (μm/m)	158	255	133	131	162	132	63	33	4
Loading in Pile (kN) (Uncorrected)	10531	16944	8834	8738	10797	8750	4201	2161	277
Loading in Pile (kN) (Corrected)	13737	13398	12889	9767	8907	8125	4201	2161	277
SPT Load Factor	**1.26**	**1.16**	**4.00**	**4.13**	**1.05**	**0.95**	**1.48**	**0.54**	**0.60**

CONSTRUCTION OF BORED PILES

Two different construction methods were adopted by the Contractor to construct the bored piles: (1) excavation under water with temporary casing provided down to the pile founding level or (2) excavation under bentonite slurry to support the pile bore with temporary casing provided to the top few metres only.

Using the temporary casing method, the soil in the pile bore was excavated by grabbing while the rock was excavated by reverse circulation drilling (RCD) as illustrated in Photo 1. Chiseling in bedrock was not allowed in the Contract to avoid cracking of the rock material around the sockets so as to ensure the quality of the sockets. The water in the pile bore would be de-silted by air-lifting and re-circulation. The temporary casings were extracted during concreting by the tremie method.

Using the bentonite method, the soil in the pile bore was excavated mainly by augering plus grabbing of some bouldery material when necessary. Chiseling was also not allowed to excavate bedrock. In lieu of RCD, the Contractor chose to use a Roller Bit Core Barrel (Photo 2) to excavate in rock to form the rock socket. The Roller Bit Core Barrel was attached to the augering rod to grind the rock around the perimeter of the barrel as the rod rotates. An annulus was formed along the socket perimeter in the bedrock. The remaining rock material at the centre were broken by a cross cutter (Photo 3) fixed to the augering rod with cutting bits at the centre and then removed by grabbing. This rock excavation method also served the purpose of forming a high quality rock socket similar to RCD. The bentonite slurry in the pile bore would need to be recirculated and de-sanded to remove the soil/ rock materials in the pile bore prior to concreting. The de-sanded slurry was displaced during concreting by the tremie method.

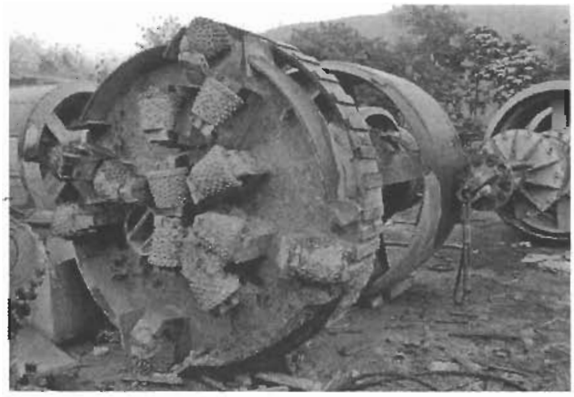


Photo 1. An RCD Head

PILE INTEGRITY TESTS

After the concrete had achieved sufficient strength, all the bored piles were subject to sonic logging to detect if there is any irregularity in the concrete and/or pile configuration. The sonic logging was carried out through the four sonic tubes installed in each of the piles. Interface coring was also carried out in the 150-mm diameter sonic tube, from approximately 1 m

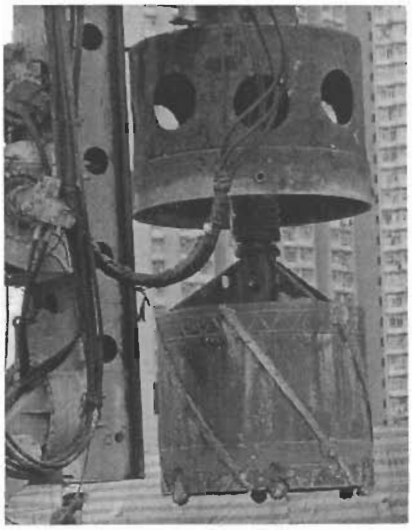


Photo 2. A Roller Bit Core Barrel

above the pile base downwards to 1 m into the founding bedrock for all the tip resistance piles, to evaluate the rock quality at the founding level and to identify if there is any soil sediments or concrete segregation at the pile base. About 5% to 10% of the constructed bored piles will also be chosen for full-length coring. Full-length coring will also be performed on piles with some irregularities revealed in interface coring or sonic logging. As requested by BD, the chosen side resistance piles will also be subject to proof-drilling at a location adjacent to the pile to verify the pile side resistance capacity by the performance of SPT.

A total of 498 sonic tests results are available at the time of writing. Some of the test results indicate irregularities ranging from minor imperfection to major loss of signal in a few piles. There is no remarkable difference in the percentage of piles that showed irregularities between piles constructed by the temporary casing method and those by the bentonite method. The sonic logs are to be further reviewed by the Contractor and the Engineer. Full-length coring will be ordered for some piles that showed major anomalies. The requirement of any remedial measures shall be determined based on the full-length coring records.

Photo 3. A Cross Cutter Head

The interfaces of 583 tip resistance piles have been cored to date. 535 of these piles were constructed using temporary casing and excavation under water whereas only 48 were constructed using bentonite. Some of these coring were full-length coring as some of the 150-mm diameter sonic tubes were damaged or doubtful coring records were obtained during interface coring.

Segregation of concrete at the pile base was revealed by the interface cores in some piles. Most of the segregation or sediment at the pile base were less than 100 mm thick. Moreover, it appears that the segregation at the pile base was observed in a larger percentage of the piles constructed using bentonite. It is therefore believed that the desilting process could be more difficult with the use of bentonite as the excavation fluid. This might have resulted in the sedimentation of soil particles or aggregates at the pile base that could be more difficult to be displaced by concrete together with the bentonite slurry. However, the number of piles so constructed may be too small to be conclusive.

Remedial Works on Piles with Interface Problem

For those piles where segregation or sedimentation were found at the pile base, the following guidelines on the remedial works have been agreed between the Engineer and the Contractor:

Table 2: Guidelines on Bored Pile Remedial Works with Interface Problem

Segregation Thickness	Remedial works	Verification Coring
0 – 100 mm	Clean-out interface with water and pressure grout through reservation pipe (inside sonic tube).	Not required unless instructed by the Engineer.
100 – 200 mm	Clean-out interface with high-pressure water jetting and pressure grout through reservation pipe plus at least one additional drill hole. The water jetting pressure should be between 20 MPa and 40 MPa to be determined on individual cases.	Required down to the pile rock interface to validate the effectiveness of the remedial works.
> 200 mm	To be proposed by the Contractor for the Engineer's review and acceptance.	To be proposed by the Contractor. Usually required.

In addition, all the remedial works would need to be supervised by senior engineers from both the Resident Site Staff and the Contractor as required in PNAP 242 (BD 2000b).

For piles with segregation at pile base up to 200 mm, the remedial works followed the guidelines tabulated in Table 2. The verification coring for those thicker than 100 mm or as ordered by the Engineer were found to be satisfactory. For those piles with segregation at pile base thicker that 200 mm, similar treatment of using high pressure water jetting and pressure grouting from two holes were also carried out. The grout pressure was held for 10 to 15 minutes to enhance grout penetration for some defective piles. These remedial works were also found to be successful as demonstrated by verification cores.

For other defects observed including the fractured rock cores and the defective concrete, pressure grouting was also carried out as remedial measures. The defective concrete or contaminated concrete was removed by high pressure water jetting prior to grouting. The verification coring after the remedial works were all satisfactory.

CONCLUSIONS

To cope with the different ground conditions along the MOS Rail alignment, particularly the large variation in rock head level, both tip resistance and side resistance bored piles are used to support the viaducts and station structures. Pile rock sockets have been designed to the acceptance of BD to support the superstructure loading together with the tip resistance of Grade III or better bedrock. Side resistance bored piles founding in CDG are designed based on the criteria on both the side resistance and tip resistance as agreed with BD.

Both the tip resistance and side resistance bored piles have been successfully constructed to the acceptable standard. The side resistance piles were all constructed using the Contractor's proposed bentonite method. Results of the first load test on a side resistance bored pile indicate that the test pile can achieve the design capacity with a factor of safety of 2.

The sonic logging test results and interface coring records indicate that there are some irregularities in the pile concrete or at the pile base that would require remedial measures. Nevertheless, all the detected defects in the constructed piles can be rectified by remedial works.

REFERENCES

Bieniaski Z. T. (1984). *Rock Mechanics Design in Mining and Tunneling*, A. A. Balkema.

BD (1995). "Foundation design – Building (Construction) Regulations 1990 – Part VI." PNAP 141, Buildings Department, Hong Kong SAR Government.

BD (2000a). "Pile foundations." PNAP 66, Buildings Department, Hong Kong SAR Government.

BD (2000b). "Quality supervision requirements for foundation works." PNAP 242, Buildings Department, Hong Kong SAR Government.

Fugro Geotechnical Services (HK) Limited (2001). "Static Load Test Report HEO to MOS Viaduct – Pier 117A, Pile No. P117A-5 - Rev. 0." December.

GCG (Asia) Ltd. (200). "East Rail Viaduct Contracts TCC200 & TCC300, Review of Requirement for Friction Pile Load Tests." Report Ref. R96/6, January.

GEO (1994). *Guide to Retaining wall Design,* 2nd Edition, Geoguide 1, Geotechnical Engineering Office, Civil Engineering Department, Hong Kong SAR Government.

GEO (1996). *Pile Design and Construction*. Publication No. 1/96, Geotechnical Engineering Office, Civil Engineering Department, Hong Kong SAR Government.

Horvarth, R. G. (1978). *Field Test Data on Concrete-to-Rock Bond Strength for Drilled Pier Foundation*, University of Toronto.

Osterberg, J. O., and Gill S. A. (1973). "Load transfer mechanism for pier socketted in hard soils or rock." *Proc., of 9th Canadian Symposium on Rock Mechanics*, Montreal.

Pells, P. J. N., and Turner, R. M. (1979). "Elastic Solution for the Design and Analysis of Rock Socket Piles." *Canadian Geotechnical Journal.*

Rosenberg, P., and Journeaux, N. L. (1976). "Fiction and End Bearing Test on Bedrock for High Capacity Socket Design." *Canadian Geotechnical Journal.*

Tomlinson, M. J. (1994). *Pile Design and Construction Practice*, 4th Edition, E& FN Spon, London.

Williams, A. F., and Pell P. J. N. (1981). "Side Resistance of Rock Socket in Sandstone, Mudstone and Shale." *Canadian Geotechnical Journal.*

Wyllie, D. C. (1991). *Foundation on Rock*, E& FN Spon, London.

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九廣鐵路大圍車站擴建之岩土工程設計

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GEOTECHNICAL DESIGN OF THE EXTENSION TO TAI WAI RAILWAY STATION

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撮要

作爲九廣鐵路公司的東區鐵路延長段的一部份，現有大圍車站擬將擴建，並將成爲連接新馬鞍山鐵路和現有東鐵的中轉站。 百泰合樂聯營工程顧問公司受九廣鐵路公司委派作爲大圍車站擴建項目之詳細設計的顧問。

本篇文章論述了與大圍車站擴建之設計有關的岩土工程方面的問題，以及對地鐵開挖和擋土墻建造工程中實測與預測之地層位移進行了總結分析。

本文還就以下問題進行了特別討論:

- 場地歷史、場地條件及設計參數;
- 在施工期間，維持現有大圍車站繼續營運所引起的施工問題，包括鄰近現有鐵路和月台的開挖方法;
- 預測及控制地層位移及沉降; 以及
- 在現有鐵路下面，設計和建造兩條新地鐵的有關問題。

GEOTECHNICAL DESIGN OF THE EXTENSION TO TAI WAI RAILWAY STATION

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INTRODUCTION

Background

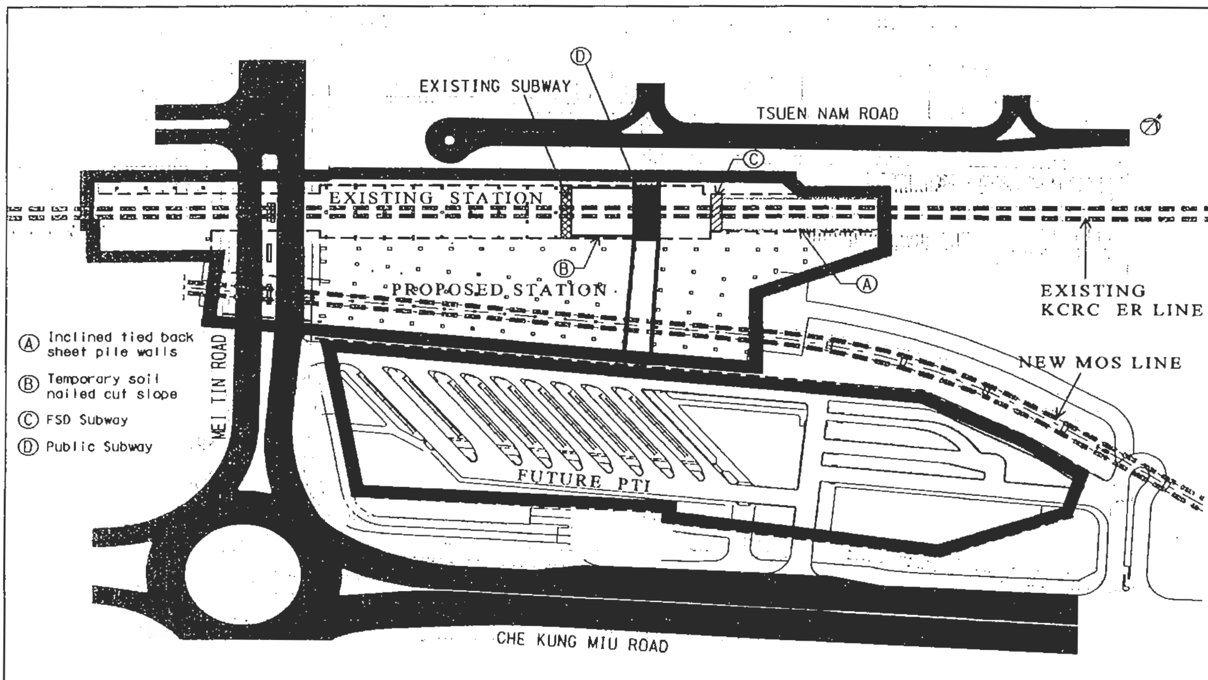
As part of the KCRC East Rail Extension, the existing Tai Wai station was to be modified and extended to act as an interchange station for the new Ma On Shan (MOS) railway at it's southernmost end, whilst continuing to serve the existing East Rail. The Babtie Halcrow Joint Venture (BHJV) - a joint venture between Babtie Asia Ltd (BAL) and Halcrow Asia Partnership (HAP) was appointed as the consultant for the detailed design of the station.

This paper describes some of the geotechnical aspects of the detailed design of the station re-development.

The Site

The Tai Wai station site is located in a broad flat lying river valley bounded by the high ground of the Lion Rock and Shing Mun country parks to the south-east and north-west respectively.

The site incorporates the current Tai Wai Station and an area of land to the east originally incorporating a bus terminus, public cycle park, private car parks and the Happy Dragon Recreation Park. The site and major geotechnical elements of the works are shown in Figure 1.



TCC400 TAI WAI STATION LAYOUT PLAN

Figure 1

[1]Senior Engineer, Halcrow China Ltd [2]Senior Engineers Representative, TCC-400, Tai Wai Station [3]Technical Director, Babtie Asia Ltd [4]Construction Manager (Ma On Shan), Kowloon – Canton Railway Corporation [5]Senior Engineer (Ma On Shan), Kowloon-Canton Railway Corporation

The existing railway is mainly on embankment. The embankment is up to 7.7m high with 30° sideslopes and is about 16m wide at the top. A railway bridge carrying the existing East Rail rail tracks across Mei Tin Road is located near the south-western end of the site.

The main structures affected by the works comprise the existing East Rail rail lines, the existing Tai Wai Station and the Mei Tin Road Bridge.

GEOLOGY

The area is underlain by coarse grained granite underlying estuarine and alluvial deposits. Marine deposits occur to the northeast of the site.

A dominant south-west to north-east trending fault system including the Lai Chi Kok – Tolo Channel Fault truncates an earlier, north/south to north-west/south-east trending fault set. Significant microfracturing of the underlying granite and variable rockhead levels have resulted from this faulting.

The general geological profile in the vicinity of the site comprises 2.5m to 5m thick fill overlying 5m to 8m thick alluvium overlying 12m to 35m thick completely decomposed granite (CDG). Localized estuarine deposits up to 2m thick are interlayered with the alluvium and a layer of cobbles is present at the base of the alluvium/estuarine deposits over much of the site. Bedrock, defined as moderately decomposed granite (MDG) or better was encountered at depths varying from 15.6m to 48.0m below original ground level. The bedrock is affected by microfracturing and hydrothermal alteration in some areas. The existing railway is constructed on a recompacted CDG embankment.

Groundwater levels vary between +4.42mPD and +1.11mPD (2.05 and 5.70m below original ground level). As the station site is situated towards the south-western end of Tolo Harbour, ground water levels are tidally affected.

Engineering properties of the various materials encountered in the site were determined from both field and laboratory test data. Engineering properties for each of the materials are summarised in Table 1.

Table 1: Geotechnical Design Parameters

Soil/ Rock	Bulk Den	Eff. Shear Strength		Und. Shear Strength	Coef. Earth Press at Rest	Elastic Mod.	Rock Mass Mod.	Pois. Ratio	Consolidation Parameters				Mass Permty	Subgrade Reaction	
	γ' kN/m^3	c' kPa	ϕ' °	S_u kPa	K_0	E_s MPa	$E_{(M)}$ GPa	ν	C_c	C_α	C_v m^2/yr	m_v m^2/kN	k m/s	Depth m bgl	n_h MN/m^3
Emb. Fill	17	0	32	-	0.47	20	-	0.4	-	-	-	-	2.0 X10^{-4}*	0~6	1.3
General Fill	17	0	30	-	0.50	10	-	0.3	-	-	-	-	-	6~20	4.4
Alluvial Clay	18	-	-	0.25p_0'	1.00	4-6 8-10	-	0.3	0.21-0.3	0.0084-0.012	2.24-104.6	0.14-1.06	1.2 X10^{-6}	below 20	10.7
Alluvial Sand.	19	5	35	-	0.43	25	-	0.3	0.16	0.0064	1.69-3.0	0.19-0.93	2.4 X10^{-6}		

Table 1: Geotechnical Design Parameters (Cont.)

Soil/ Rock	Bulk Den	Eff Shear Strength		Und Shear Strength	Coef Earth Press at Rest	Elastic Mod	Rock Mass Mod	Pois Ratio	Consolidation Parameters				Mass Perm.	Subgrade Reaction	
	γ' kN/m^3	c' kPa	ϕ' °	S_u kPa	K_0	E_s MPa	$E_{(M)}$ GPa	ν	C_c	C_α	C_v m^2/yr	m_v m^2/kN	k m/s	Depth m bgl	n_h MN/m^3
Cobble Layer	18	0	35	-	0.43	50	-	0.3	-	-	-	-	-		
CDG	19	5	36	-	0.41	10 to 80	-	0.3	-	-	-	-	1.6 X10^{-6}		
Grade III/II Rock	23	100	40	-	0.36	-	15	0.2	-	-	-	-	-		

Notes: 1 p_o' = Effective vertical stress

2 E_s, E_m, ν values based on Table 9 of Geoguide 1 (GEO, 1993) and SPT 'N' values.

3 $C_a/C_c = 0.04 \pm 0.01$ (Mesri and Choi, 1984).

4 Permeability values, k, are determined based on falling head permeability tests carried out during ground investigation. Values with * are determined using correlation with particle size grading using the following equation: $k=10^{-2}(D10)^2$ (m/sec)

5 n_h values based on Table 10 of Geoguide 1 (GEO 1993) and SPT 'N' values.

FOUNDATION DESIGN

Shallow foundations were deemed to be impractical due to the soft, slightly organic clays and loose sandy silts present beneath the site. Piled foundations were therefore adopted.

The station foundations were designed as bored cast-in-place concrete piles, end bearing on rock. These were chosen as the most suitable foundation system due to the high loads to be carried, their flexibility of construction, the need to minimise construction noise and vibration in a built-up area, and the potential for damage to existing piles and structures from percussive piling. The design and construction of bored piles followed the requirements of the Buildings (Construction) Regulations and Practice Notes (PNAP 66 and 141) issued by the Hong Kong Government. Bored pile foundations of between 1.2m and 2.5m diameter, belled out in some cases to between 1.5 and 2.5m, were adopted. Rockhead varies across the site, being generally deepest at the south-western and north-eastern ends of the station. Piles were therefore predicted to vary between 14m and 36m in length.

A presumed bearing capacity of 5 MPa (from Table 1 of BD Practice Note 141) was taken for piles founded on Grade III decomposed granite with a total core recovery of 85% and a minimum unconfined compressive strength (UCS) of 25 MPa (or point load test strength of 1 MPa). No skin friction in the transported soils or saprolites was allowed for in the pile design. Piles were designed to be socketed a minimum 0.5m into rock in accordance with PNAP 141 requirements.

Effect of New Piles on Existing Structures

In the areas of the existing station, new piles were required to be installed close to existing H piles supporting the reinforced concrete platform boxes. Working loads for these piles were given on as-built drawings as 1490 kN to 2000kN per H pile. Back calculation of the H-pile working load indicated that these working loads would be achieved at a depth of about 14m below ground level (bgl). Since rockhead beneath the Station varies between 16m and 35m bgl it was believed that the existing H piles would be obtaining most of their load capacity from skin friction within the saprolites. It was feared therefore, that ground loss occurring during pile installation could lead to a loss of skin friction on the existing H piles, which in turn could lead to additional loads being transferred to the H pile base resulting in settlement of the piles and station platforms. To minimise this risk, it was specified that no new piles should be installed within 3 pile diameters of the existing H piles. In addition, any piles constructed within 3 perimeters of the existing H piles would require special measures such as bentonite to minimise lost ground during pile installation.

Reverse circulatory drilling (RCD) rather than chiseling was specified for forming rock sockets to minimise vibration and maximise the quality of the pile/rock interface at the pile base. A "Koden" ultrasound system was specified for verifying the shape of any bellouts prior to concreting.

At the southern end of the structure, bored piles were required through the existing rail embankment for the foundations of a permanent RC retaining structure. To eliminate lateral forces on the piles from the embankment fill and underlying soft soils, these were designed with a permanent casing over their upper part, to form a gap between the embankment fill and the installed pile.

Retaining Structures

Permanent RC retaining structures were required beneath the existing ground bearing and piled station platform boxes for some of the internal walls for the new station, and for a new access on the southern side of Mei Tin Road.

The walls beneath the station platforms were required to have their external face constructed flush with the existing station platform walls. For the existing piled station platform areas, it was possible to excavate the existing embankment slope beneath the platforms to allow the construction of conventional L-shaped retaining walls in front of the cut slope. To achieve this, a 60° soil nailed temporary cut slope was proposed. A factor of safety of 1.4 was adopted for the slope design to ensure safety and minimise movements that could affect the existing piled platforms and rail tracks. In addition, sheet piles were installed along the slope toe to prevent deep seated failure.

It was recognized that excavation beneath the existing piled station platforms would remove support from the underside of the platform box ground beams. The structural capacity of the ground beams was therefore checked to confirm that they were adequately reinforced for the temporary case. Permanent support for the beams was provided by the completed permanent RC walls. Due to the restricted height availability and enclosed space beneath the existing platforms, it was envisaged that the floor screed of the overlying platform boxes would be required to be broken out to allow backfilling behind the wall, or alternatively mass concrete would be used for the upper part of the backfill. An alternative of using hydraulically placed sand is however currently being explored.

Where the existing station platforms are ground bearing rather than piled, removal of the embankment sideslopes requires that the platforms be supported by a temporary retaining structure. The configuration of the existing platforms, plus the requirement that the new walls be flush with the existing station walls, required the use of a novel temporary support system. Temporary contiguous pipe pile retaining walls, inclined inwards at 5° were proposed along either side of the station. As the embankment was excavated, steel ties would be installed through the railway embankment and stressed to a pre-determined level before excavation proceeded to the next stage, Figure 2. The initial design also used external struts. On completion of the permanent wall the steel ties and struts are removed, but the temporary pipe pile wall remains in place. Although the contractor was not obliged under the contract to adopt the proposed system, he nevertheless chose to do so. Sheet pile walls rather than the envisaged pipe pile wall system were however used and external struts were replaced with higher stressing in the tie bars.

The use of sheet piles, rather than pipe piles, raised the possibility of a cofferdam effect being formed by the sheet piles and one of the proposed new subways, which could lead to saturation of the embankment fill. To mitigate this effect, drainage holes, lined with geotextile filter, were made through the sheet piles and a drainage system incorporated into the back of the permanent RC wall.

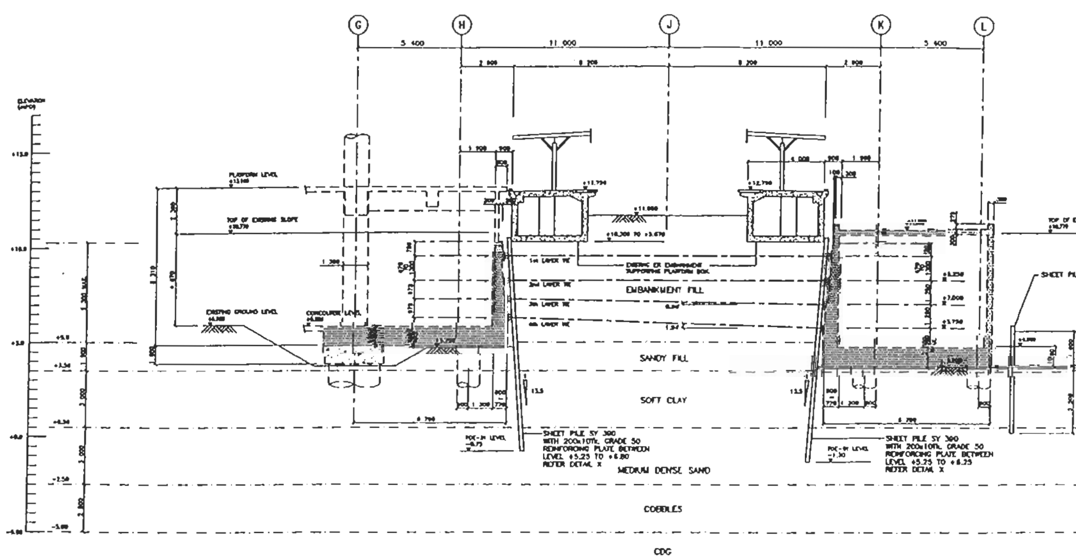


Figure 2: Typical layout of shoring for retaining wall construction

Owing to the difficulty in predicting movements of the wall and the adjacent structures, a finite difference analysis was carried out (using FLAC) to model the excavation and support sequence. This indicated that, using tie bars and external struts, maximum lateral movements at the base of the excavation and associated settlements of the platform boxes could be reduced to less than 10mm. However, to remain within the specified KCRC settlement and cant tolerances for the rail tracks (5mm and 2mm over 5m respectively), it was expected that it would be necessary to re-tamp or re-ballast the rail tracks periodically during the excavation works. A revised FLAC analysis was also undertaken to model the contractor's support system (without external struts). This indicated maximum movements of the tracks of +/- 7mm and a maximum inwards movement of 35 mm.

As stated above, the contractor decided to forgo external strutting of the walls in favour of an increased pre-stress in the tie bars (raised from 100 kN/tie to up to 1476 kN/tie). After installing the 1st row of ties (stressed to 635 kN), excavating to the second level and applying the full 1476 kN pre-stress to the 2nd row of ties, the sheet piles moved inwards by 45mm, moving the platforms and tracks laterally together by up to 25mm on each side. An urgent review was undertaken by the RSS and contractors staff and it was decided that an observational approach to stressing of the ties was a safer way to proceed for the excavation works. Consequently, the ties for all remaining excavation stages were stressed to 50% of their design load and the movement of the sheet piles monitored. Additional stress would subsequently be applied if excessive outwards movement was recorded. In fact all remaining stages of excavation proceeded smoothly with no further unexpected displacements.

Subways

Two new subways were required to be constructed beneath the existing Tai Wai Station platforms. One subway, (the Fire Services Department (FSD) subway) was required for utilities and emergency access by the Fire Brigade and a second subway (the Public Subway) was required for a new main station entrance and 24-hr public access between Tsuen Nam Road and the Public Transportation Interchange on Che Kung Temple Road. Since the FSD subway passes underneath the existing ground bearing platform boxes and the Public subway passes beneath the piled platform boxes, each subway presented similar, but also different, geotechnical challenges.

Construction of both subways had the potential to generate significant movements of the overlying platform boxes and/or rail tracks. A support system was therefore needed to minimise these movements. A temporary support system comprising a steel box formed of horizontal pipe piles within which excavation for the subways would take place was devised. The pipe piles were to be supported by internal steel frames, installed as excavation progressed, Figure 3.

Trial pits carried out in the embankment at the location of the subways indicated that the embankment was constructed of compacted CDG with a variable relative compaction of 85% to 90% (locally 80% to 95%). The embankment also contained occasional cobbles and boulders of up to 400mm diameter. Laboratory shearbox tests indicated shear strengths of the embankment material of about c'=0 kPa, ϕ'=32°.

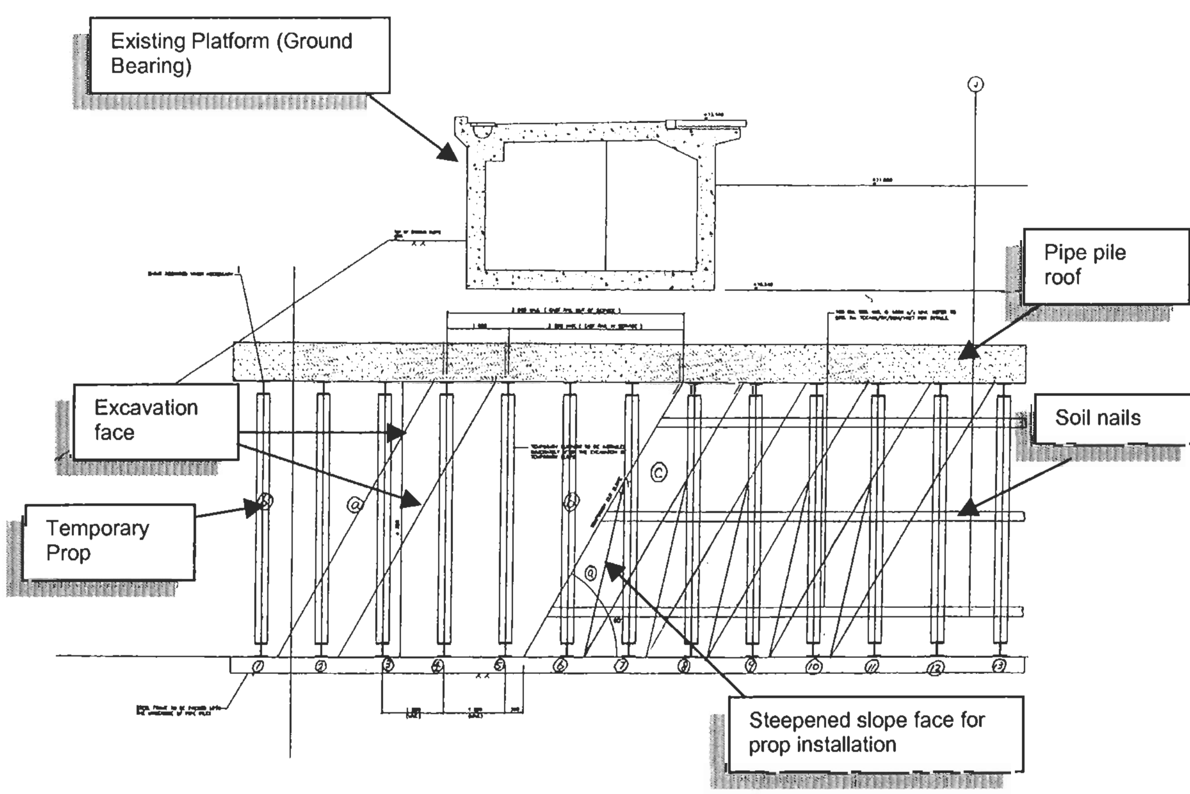


Figure 3: Temporary Works for subway excavation

Movements above the box were expected to result mainly from:
i) settlement from ground loss during installation of the pipe piles,
ii) settlement due to structural deflection of the roof pipe piles,
iii) deflection of the temporary support system.

Structural deflections of the internal frames and roof spiles during subway excavation were calculated for various unsupported span lengths. The possibility of grouting the embankment soils to allow steeper cut faces to minimise the unsupported span was also considered, but the fines content of the fill (typically >20%), combined with a lack of overburden for pressure grouting, appeared to make this unworkable. Further, the variable nature of the fill was likely to result in claquage of the grout. Instead, steel or fiberglass soil nails were proposed to reinforce the soil face during excavation. In this way, the excavated face could be maintained at a 60° angle such that calculated maximum deflections of the pipe pile roof were less than 10mm. Again, the contractor was not obliged to adopt the proposed support system but in fact chose to do so, but with somewhat larger diameter pipe piles than envisaged at design stage.

To ensure the safety of the operational railway, the installation of the subway pipe piles and subsequent excavations were required to be carried out in stages. A typical sequence is indicated below:
i) The roof pipe piles of the pipe pile box were installed followed by those for the sides of the box. Pipe piles were allowed to be installed only up to the inside edge of the overlying platform boxes during normal working hours. Installation beneath the rail tracks was restricted to non-traffic hours (1:30 to 4:30 am). After each non-traffic hours working shift, the rail track level is checked and an all clear signal given at the end of the shift to KCRC operational staff to confirm that it was safe to recommence train operations.
ii) After installation of the pipe piles, temporary soil nails were installed inside the pipe pile box.
iii) Grouting above the crown of the subway box was carried out by tube-a manchette to improve the ground and minimise water flow into the subway during excavation.
iv) Excavation commenced, cutting away the soil nails and installing internal frames as excavation proceeded.

FSD Subway

This subway, to be used as an emergency access and utilities route, passes beneath the overlying station platforms which are ground bearing on the existing embankment. Limiting settlements of the overlying platforms as well as the rail tracks was therefore a key design element. From as-built records, the platform box was known to have been constructed of 12m long, 4m wide and 2.7m deep, concrete box segments. It was believed therefore that these would be likely to reticulate to accommodate settlement movements. A settlement profile based on the use of 200 mm nominal diameter pipe piles, was determined for the subway. Maximum settlements and angular distortions were determined which, based on the empirical damage criteria proposed by Rankine (1988), indicated that negligible to slight structural damage could be anticipated. In fact, the contractor chose to install 600 mm diameter pipe piles for easier installation and better structural support. Settlements were therefore larger than originally anticipated, see Table 2.

Table 2: Estimated and Actual track settlement above FSD Subway

	Estimated (200 dia pipe piles)	Estimated (600 dia pipe piles)	Actual
i) Completion of pipe pile installation	-8 mm	-71mm	-48 mm
iii) Completion of tube-a-manchette grouting	Not considered	Not considered	-36 mm
ii) Completion of installation of soil nailing	Not considered	Not considered	-40 mm
iv) Completion of Excavation	-23 mm	-83 mm	-60 mm

-ve = settlement

Despite actual settlements being significantly larger than anticipated for the original design, the real-time monitoring system enabled a continuous check to be kept on the differential settlements between rail tracks (rail cant) to ensure that train and passenger safety was never compromised. When necessary, the tracks were re-levelled using a combination of hand tamping of the rail tracks and periodic mechanical re-tamping. Other than a 10mm step at the junction between the ground bearing and piled platform boxes, no signs of damage of the platform boxes were observable following completion of the subway works.

Public Subway

Construction of a new station access (the Public Subway) was complicated by the presence of H piles supporting the existing Station platforms, Figures 5a and 5b.

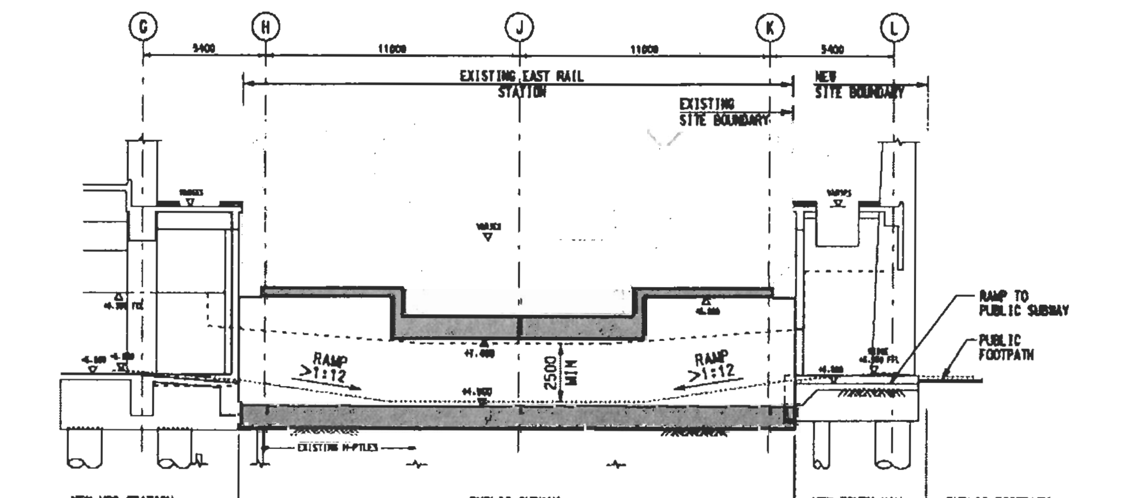


Figure 5a: Section through Public Subway

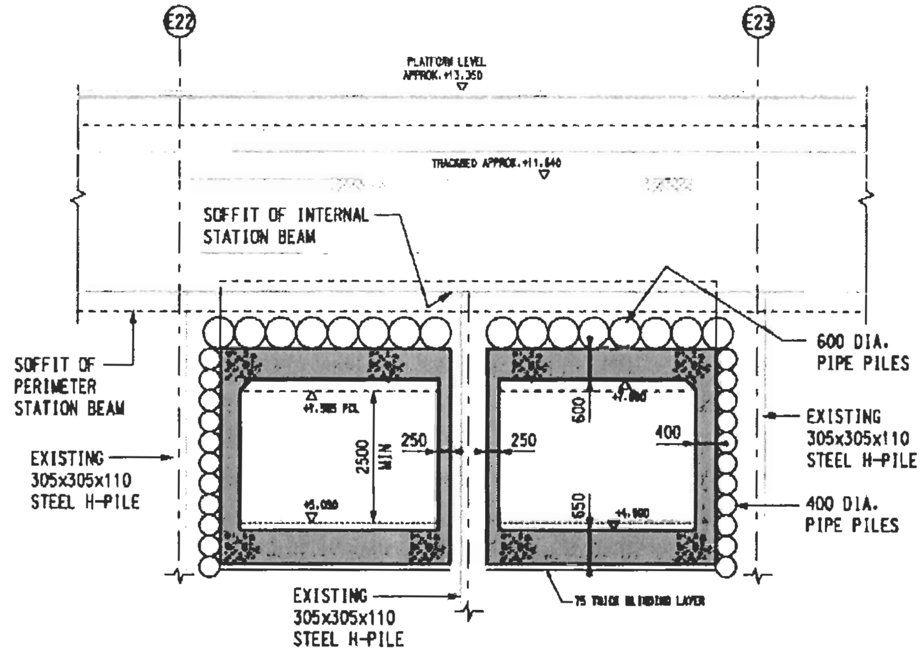


Figure 5b: Elevation of Public Subway

Installation tolerances of 1 in 100 were allowed for in the design for deviation of the pipe piles during installation. Originally, 150 mm diameter pipe piles were proposed to minimise settlements. However 600mm nominal diameter pipe piles were adopted by the contractor for the roof piles with 400mm diameter pipe piles for the walls. Excavation proceeded in a similar manner to the FSD Subway. Since the horizontal pipe piles could not pass through the areas occupied by the central raking H piles, a small 700 mm wide section of the subway roof was unable to be supported by the pipe piles. Support of this section during was achieved by welding steel plates to the adjacent roof pipe piles as the excavation progressed.

Settlements of the overlying platforms and rail tracks were continually monitored during the excavation process. As for the FSD subway, due to the increased size of pipe piles (400 diameter for the walls of the temporary box and 600 diameter for the roof) actual settlements of the tracks were greater than anticipated as indicated in Table 3.

Table 3 Ground movements of the Rail Tracks Above the Public Subway

	Estimated (200 dia pipe piles)	Estimated (600 dia pipe piles)	Actual
i) Completion of pipe pile installation	-8 mm	-71mm	-49 mm
ii) Completion of installation of soil nailing	Not considered	Not considered	-54 mm
iii) Completion of tube-a-manchette grouting	Not considered	Not considered	-54 mm
ii) Completion of Excavation	-23 mm	-78 mm	-88 mm

-ve = settlement

Despite these recorded settlements, little or no damage was evident in the platform structures and periodic re-leveling of the rail-tracks ensured the continued safety of the railway. The station platforms above, being on piled foundations, were not affected by the movements.

Protection of Existing Structures

A primary requirement of the design of the works was to protect the existing railway lines, station and bridge structures, roads and utilities from damage during the construction works. The primary causes of damage were identified to be from construction induced vibrations and settlements.

Owing to the potential for damage and settlement particularly of the existing Tai Wai Station, the use of driven piles, percussive sheet piling and percussive installation of pile casing were specifically excluded from areas close to existing structures. The Contractor was allowed to use chiseling techniques provided that vibrations were demonstrated not to exceed specified allowable values. Along the existing station platforms, the contractor used the a non-vibratory sheet piling technique (which pushes sheet piles in place without vibration or percussion) for the temporary retaining walls adjacent to the existing station. The adoption of bored cast-in-place techniques for the piles also minimised vibration.

Protection of existing structures was based on a careful review of the contractor's method statements and real time monitoring of movements during the works. "Alert", "Action" and "Alarm" response values were specified for movements of each affected structure type, so that appropriate actions could be taken in the event of unexpected movements being recorded. Details of the monitoring systems used are described by Tse et. al. (2002).

Monitoring instrumentation was installed along the tracks, on the station platforms on roads and utilities and on temporary works, to monitor ground movements and stresses in structures during the construction works.

Horizontal control stations comprising piled plinths were installed to ensure that an accurate baseline level could be determined as a reference for other instruments. Surface settlement markers were installed on the existing station platforms and around the perimeter of the site. Monitoring of the tracks was via electrolevel tiltmeters and an automatic deformation monitoring system, Tse et. al. (2002) which provided real-time monitoring data to an accuracy of better than 1mm. Tiltmeters were also installed on overhead electrical masts to measure their deflections.

Vertical inclinometers were installed at 10m spacings on the inclined temporary sheet pile walls along the existing station platforms and stresses in the tie bars were monitored using lift-off techniques on selected ties. Horizontal inclinometers were installed inside roof pipe piles forming the subway temporary supports.

All instruments were monitored against a specified table of Alert, Action and Alarm values which were reviewed and modified as necessary in light of actual movements and performance of the ground.

CONCLUSIONS

Construction of the Tai Wai Station extension presented several major geotechnical challenges, specifically related to monitoring and controlling ground movements adjacent to and below the existing station and rail lines. The following critical geotechnical items were identified:

i) Limiting ground movements adjacent to existing station platform boxes and railway lines during excavations for construction of subways and retaining walls.
ii) protecting existing structures during installation of piles and temporary works,

iii) ensuring stable temporary cut faces of tunneling works during subway construction works,
iv) monitoring and controlling movements of the overlying rail tracks
v) providing stable temporary cut faces beneath the existing piled platform boxes during construction of a permanent RC retaining wall beneath the platforms.

The geotechnical design endeavored to ensure minimal disturbance of the existing railway and structures. In certain cases, such as along the excavated embankment, and during construction of the subways, movements were inevitable. In these cases the possible ranges of movements were assessed and Alert, Action and Alarm levels were developed to indicate if the structure or ground behaviour deviated significantly from that expected. The effects of the construction works could then be monitored against these values and corrective action taken as required. For the installation of an inclined, tied sheet pile retaining wall, it was found that an observational approach to the works was required in order to control ground movements during construction.

REFERENCES

Geotechnical Control Office (1986). *Geology of Sha Tin. The Hong Kong Geological Survey Memoir No. 1.* Geotechnical Control Office, Hong Kong, pp 11-16.

Geotechnical Control Office (1987). *Geotechnical Area Studies Programme Report II. Central New Territories.* Geotechnical Control Office, Hong Kong, pp 15-25.

Geotechnical Control Office (1993). *Guide to Retaining Wall Design. Geoguide 1.* Geotechnical Control Office, Hong Kong.

Institution of Civil Engineers (1996). The Observational Method in Geotechnical Engineering. Geotechnique, Vol 44 No.4

Tse, C.M., Chan P., Chan, W., Tyson, J., and Yin, K.K.. "Geotechnics of the KCRC Ma On Shan Rail." Proc., 22nd Geotechnical Division Annual Seminar, HKIE Hong Kong.

九廣鐵路大圍維修中心之岩土力學之方面

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GEOTECHNICAL ASPECTS OF THE KCRC TAI WAI MAINTENANCE CENTRE

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撮要

在建造中之『九廣鐵路大圍維修中心』是『九廣鐵路公司馬鞍山線』之鐵路車輛維修車廠。維修中心之上面建有平台上蓋，上蓋用作發展高層住宅物業；維修中心之面積大概是 6.5 公頃，建築物由一共 422 支『大型灌注樁』鑽土或樁柱及 1109 支『預鑽式 H 樁柱』支樑。一部份樁柱所在之土地的地質是受到『荔枝角及吐露港斷層』之影響範圍之內。

本文提供有關『大圍維修中心』建造工程中之兩項土力工程方面的論述：-

(a)　該維修中心之樁柱工程之概述，比較則重於在受到複雜地質 ，石質深層風化及受地下斷層影響地方之樁柱工程遇到的設計及建造之問題。

(b)　這個工程所得到之經驗。

Geotechnical aspects of the KCRC Tai Wai Maintenance Centre

K K Yin[1], Simon Kemp[2], Ian Askew[3], Charles Chor[4]

Abstract: The KCRC Tai Wai Maintenance Centre (TMC) is a fully decked depot, which serves the Ma On Shan Rail (MOS). Built as a closed train and railway maintenance centre with a top-side development, the footprint of the maintenance centre will cover an area of approximately 6.5 hectares and be supported by 422 large diameter bell-out bored piles and 1109 pre-bored H-piles. Some of the piles are affected by the deep weathering zones associated with the Lai Chi Kok Tolo Harbour fault.

In this paper, the authors provide an overview of two major geotechnical aspects of the Tai Wai Maintenance Centre construction works, (a) general pile construction for the maintenance centre and in particular the design and construction concerns of piles in the complex geology and deep weathering areas associated with the fault, and (b) the lessons learnt from this particular site.

Introduction

The KCRC Tai Wai Maintenance Centre (TMC) currently under construction shall serve as a maintenance centre for the proposed KCRC MOS line. The MOS rail is a double-tracked electrified domestic railway serving Ma On Shan from Tai Wai to Wu Kai Sha, figure 1 refers. It is 11.4 km in length and comprises of nine stations.

Ove Arup and Partners HK Ltd are the engineer with Gammon-Skanska Ltd as the main contractor for the HK$ 1.3 billion contract. The Tai Wai Maintenance Centre (TMC) covers an approximate footprint of 6.5 hectares of land that has been gazzetted for the construction of a railway depot. The top-side development is reserved for residential towers blocks, car parking and other associated facilities in the form of club houses and schools, figure 2 refers. The maintenance centre will accommodate a full range of facilities for the cleaning and maintenance of the MOS rail rolling stock and the maintenance of the railway infrastructure housed within a series of buildings of various sizes beneath a podium level. The buildings are generally single storey but due to site constraints some mezzanine levels have been introduced where appropriate to utilise the available space. Train access to the maintenance centre from the presently being constructed Tai Wai Station, just north of the maintenance centre, will be from the southern overrun and tail tracks located alongside the existing East Rail alignment and supported on ballast back-filled retaining walls cut into the existing East Rail embankment. Approximately 1.5 km of retaining wall structure will be

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constructed along the existing East Rail embankment with retained heights ranging from 3.4 m and 14 m.

The residential towers on the podium level above the maintenance centre will form part of the future topside development. Appropriate loading requirements have been allowed in the maintenance centre foundation design to accommodate the future development.

GROUND INVESTIGATION

Two ground investigation contracts have been awarded to establish the geological profile of the site. The first was carried out in the feasibility studies of the MOS Rail by another consultant whilst Arup defined the second in the detailed design stage (TDD500) to establish and reconfirm the ground stratigraphy of the former. Both ground investigations showed the solid rock geology at some 45m below ground level, figure 3 refers.

In the second ground investigation contract, gravity survey was introduced to substantiate the accuracy of the conventional drillholes. In addition to this, the gravity survey allows the end-user a generalise idea of the topography of the solid geology.

However, a few localised zones of contrasting profile arose from the contract pre-drill exercise in which the solid geology profile exhibited sharp variation in level. The 'cliff ' profiles agree with the Hong Kong geological maps, which confirmed that the Lai Chi Kok - Tolo Harbour fault bisected the site, which was published in the geological map. The few localised zones of contrasting rock-head profile are summarised on figures 4 and 5.

GROUND CONDITIONS

Geological Setting and Structure

The site lies within the southwest to northeast trending Sha Tin Valley. The Shing Mun River flows just to the south of the site, towards Tide Cove to the northeast. The site area is on the northwestern side of a flat alluvial plain at the base of the hills, which rises up towards Tai Mo Shan.

The geology of the Sha Tin Valley consists of a basement of coarse grained granite with a weathered profile above which is overlain by recently deposited alluvium (river deposits) and fill. This relatively simple geological sequence is complicated by a tectonic history of folding and faulting in the area. The axis of the Sha Tin Valley is also the axis of an anticlinal fold in the basement rock. The older volcanic rocks form the flanks of this fold and the younger igneous rocks of the Sha Tin Pluton have intruded the core. The upper part of this anticline has now been eroded; leaving the geological exposure that is now reported on the geological map.

In addition to this folding the area has been affected by some major faulting. The most significant fault is the Lai Chi Kok to Tolo Harbour Fault (Tolo Harbour Fault) which trends northeast along the axis of the anticlinal fold. This major regional tectonic fault passes immediately beneath the site and has been associated with significant strike slip movement (i.e. right to left, rather then up and down movement). A northwest to southeast trending set of faults is also seen on both the geological map and as photo-lineaments on the aerial photographs.

Solid Geology

The coarse grained granite of the Sha Tin Pluton forms the geological basement to the entire site. This rock type, which is well understood as an engineering material, consists of three major minerals; alkali feldspars (20%) which are generally pink, plagioclase (20%) which is generally white and quartz (60%) which is generally clear. Very minor amounts of black speckles caused by the mica mineral biotite also occur.

The elevation of engineering rock varies from +20 mPD on the western side of the site to between -30 to -40 mPD at the northeastern end of the site. This variation in the rock head level is a function of both the proximity to the hills on the northwest side of the Sha Tin Valley and the geological structure (faults and folding) within the basement rock. The initial findings from this model suggest that two "valleys" in the rockhead profile intersect at the northeastern end of the site. These "valleys" trend northeast to southwest and southeast to northwest and correlate approximately with the Tolo Harbour Fault. A third minor "valley" in rockhead trending south-east to north-west can also be identified.

The general weathered profile within the granite varies in thickness from about 5 to 10 m adjacent to the hills on the western side of the site, to 20-25 m towards the centre of the Sha Tin Valley and towards the northeast of the site. The depth of the weathered profile is generally controlled by the original permeability of the intact rock mass due to folding and faulting, intrusion, erosion etc. Therefore, where the original rock was highly fractured by folding and faulting, will tend to be where the deepest weathering will now be found.

The Sha Tin Granite is one of the older granite masses to be found in Hong Kong. As such it could be expected that it have been intruded by more recent periods of igneous activity. Minor intrusions of rhyolite, monzonite and basalt have been observed in the surrounding hills but only basalt has been identified in the existing drillholes on the site. Minor intrusions of this kind would likely be dykes and could be expected to be associated with alteration of the surrounding granite, which would probably have a detrimental affect on its engineering properties. Contact metamorphism locally to the intrusions may alter the rockmass and potentially increase the clay content. In addition the material locally may be more susceptible to weathering.

PILE DESIGN

In total, 422 bored piles with diameter ranging from 2.5 to 2.8 m with maximum bell-out of 4.2 m were designed as the foundation piles for the residential towers. The bored piles, with depths up to 80m, were designed to sustain approximately 70MN under the gravity loads. In addition, 1109 prebored H piles were also designed to resist the loading from the podium car park and other associated facilities packaged under the top-side development, figure 6 refers. The prebored H piles were each designed to support approximately 5MN. Shear studs have been specified at the socket length to optimise the bonding between the steel/grout interface.

Faced by the fast-track programme and the high numbers of foundation piles, it was essential to carefully plan the works and optimise resources during the construction stage. The Engineer and the Contractor have worked closely to agree the founding levels in an expedient manner.

Design methodologies were refined once the complete ground conditions were revealed in the pre-drills at the pile's location. The pre-drills identified some areas of localised deep weathering not previously found under the ground investigation contracts and approximates to the Lai Chi Kok-Tolo Harbour Fault and in the southeast corner of the site, Figure 5 refers.

Inter-bedded seams of highly weathered rock caused the lowering of PNAP141, Grade III rock-head levels of isolated piles and in some cases lowered the founding levels of adjoining piles and pile groups. To determine pile founding levels, the methodology outlined in the form of flow chart was followed. These methodologies were submitted to the Buildings Department for agreement.

Once the predrill record are made available, a contour plot was prepared to give a three dimensional relative position plot to ascertain the need for re-design for over-stressing by load spreading. If the rockhead profile dips critically or sharply, then there is a need to assess the stability of the 'rock' cliff. The assessment of rock cliff stability takes into account any adverse dipping angles that are likely to contribute planar, wedge and toppling failures. For the purpose of identifying the various failure modes, the TCC500 approach, adopted a conservative route by assuming the worst credible scenario. Acknowledging this approach may somewhat is on the onerous side, it reduces the time needed for interpretation of jointing from by impression packer tests and the associated costs.

Once the founding level were defined, a structural analysis was carried out to ascertain the associated differential movements between pile caps. In TCC500, the approach was to have all piles underneath the pile cap founding at a similar level to avoid the risk of compatibility under a single pile cap.

In addition, the foundation design was optimised by forming under-reams in rock of lower quality then that required for end-bearing. In typical Hong Kong practice, the under-reaming process will only commence effectively at 300 mm below the lowest inferred rockhead and with the full ream at the toe of the pile. Faced with the layers of seams within the solid geology and considering the good workmanship, the under-reaming process in TCC500 commenced within the Grade III/IV medium and with a toe in the Grade III material. This mitigation measure for which under-reaming is made allows more flexibility in the determination of founding level and the nominal 5m of Grade III material beneath the toe of the pile.

In certain areas of the site where the solid geology exhibited preferential or highly weathering grades, the cost effectiveness of utilising the Grade III/IV socket friction was more attractive. The choice of utilising socket friction within the Grade III/IV environment eliminates the risk of finding Grade III material within a zone of highly weathered material and also saved the construction time.

The above procedure applied to both prebored H pile and bored piles with the exception of under-reaming.

FOUNDATION CONSTRUCTION

The contract for Tai Wai Maintenance Centre was awarded to Gammon Construction Limited (now Gammon Skanska Limited). The tender value of the piling element of the works was about HK$ 400 million and the foundations were to be completed in 15 months. Practice to ensure a close working relationship between the stakeholders of the project was introduced on the project.

To complete the foundations within the required period it was necessary to concrete up to 14 bored piles per week. Up to 40 piles were progressed concurrently. Resources employed included 33 crawler cranes, 10 clam shell grabs, 16 reverse circulation drilling machines, 8 casing rotators/vibrators and 250 site operatives.

Typically the pile construction sequence involved rotating or vibrating a temporary steel casing into dense highly decomposed rock or rock whilst progressively removing the loose soils inside the casing with a grab under a head of water. Vibrators were used to sink casings through loose soils while for hard ground rotators were employed. For pile construction within the KCRC 30 m protection zone, the installation of temporary casing with least disturbance to the existing East Rail embankment was achieved through a series of hold points and due monitoring of the instrumentation on the KCRC tracks.

Excavation in the dense soils and rock required the use of reverse circulation drilling machines. On reaching the founding level an under-reaming tool was used to form the pile bell-out and a koden test was conducted to check the geometry of the base. The reinforcement cage was installed and the pile base was then cleaned using airlifting. Concrete was placed using a tremie-pipe from a hopper feed. Further quality checks including concrete coring, interface coring and sonic tests were then undertaken.

For deep piles or in ground comprising of mixed rock and soft soils temporary casings were telescoped, i.e. casings of progressively smaller size were used to reduce the total soil resistance for casing installation and removal. Up to two temporary casings were telescoped for the deep piles.

TCC500 LESSONS LEARNT

Koden Test

Koden testing provided a valuable insight into the stability of bell-outs constructed in highly fractured rock at this site. The geometry of the constructed bell-outs was shown to consistently match the design requirements. In TCC500, the Koden tests (Figure 7 refers) have been carried out to confirm the approximate geometry of the RCD bell-out diameter and the verticality of the pile shaft. Because of the relatively small print out size, the associated scaling issues and in some cases the blurred image the koden test is only an approximate method of checking pile bell-out diameters and pile shaft verticality. However, the black and white imagery clearly identifies a verticality trend and the maximum diameter of the bell-out. Measurements taken form the pile image should be evaluated within the tolerance of the test print out which invariably means that a degree of flexibility must be allowed when assessing compliance of the pile bell-out diameter and verticality of the shaft. The koden test effectively checks the bored pile geometry.

Tremied Concrete Strength.

Concrete design strength of 36 N/mm² was specified for the bored piles. This corresponds to 45 N/mm² concrete with a 20% reduction in strength for tremie placement under water as required by the Building Construction Regulations (1995). From strength tests conducted on cores, summarised on Figure 8, the as-placed strength of the concrete in the piles was in the range of 50 to 60 N/mm². It is suggested that with the advances in concrete technology, installation techniques and quality procedures the need for a tremie reduction factor should be reviewed. Omission of a tremie reduction factor would allow direct cost savings in reinforcement. Additional benefits would be improvements in safety and buildability by reducing the weight of reinforcement cages. Construction quality could also be improved by having less reinforcement to obstruct concrete flow.

Additional Pre-drilling in Areas of Complex Geology

In areas of complex geology where seams of grade III/IV rock were intermittent within relatively thick design grade III rock or where a sudden change in design grade III rock was identified between adjoining piles additional drill-holes were excavated through the base of the pile. This practise was another attempt to raise founding levels and to reduce the length of piles during construction. As common practise samples of rock were taken at 500 mm centres to give an indication of the change in weathering grade below rock-head. This again increased confidence in the founding levels proposed for the contractor and the resident site staff.

Triple Barrel Interface Core Retrieval.

Interface cores have proved very good interface contact between the bell-out pile base and the founding rock material, (refer figure 9). Core taken across the interface was generally sampled using a triple core barrel to reduce core loss and to increase the chance of recovery if poor rock grade was identified below the pile. Lessons learnt during interface core sampling is to use the best equipment to optimise the recovery of pile concrete and founding material to prevent difficulties in proving pile compliance if the recovery is not sufficient.

Gravity Survey

As one of the many geophysical survey techniques available currently, gravity has been proposed and implemented in TCC500. The technique does allow the average 'topography' to be defined and that provided the flexibility to the Engineer in estimating the length of piles. Despite the merits of providing the average rockhead profile, the technique is unable to define areas where the contrasting density differs marginally, for example those of Grade III/IV and III.

Close Co-ordination

Throughout this contract the resident site staff and the contractor have maintained a good relationship to progress construction and to develop a common goal to reduce the inherent difficulties of pile construction. In areas of complex geology both parties have been proactive in reducing pile lengths for founding level proposals and on site during construction to check the consistency of the proposals. The working relationship has been of paramount importance to ensure efficient pile design and quality of construction.

CONCLUSION

The geology of Tai Wai Maintenance Centre is considered complex in the locality of preferential weathering and the Tolo Harbour Fault. With the exception of the localised area of preferential weathering and faulting, the geological model derived from the ground investigation data and the published geological map seem to provide a reasonable consistency of the underlying solid geology.

The fast-track programme requires an optimisation of the contractor's resources and the engineer to work closely to meet the construction deadline. During the construction, efforts were made to arrive at a timely solution when it matters most.

One of the major lessons learnt from TCC500 is the consistently high concrete strength derived from concrete coring. The results and the statistical distribution reflected advancement in the concrete industry and that the downgrading of concrete strength due to tremie effect should be given proper attention.

REFERENCES

Buildings Department (1995). *Practice Note for Authorised Persons and Registered Structural Engineers (PNAP) 141 – Foundation Design Building (Construction) Regulations 1990 – Part VI.*

Buildings Department (2000a). *Practice Note for Authorised Persons and Registered Structural Engineers (PNAP) 66 – Piled Foundation.*

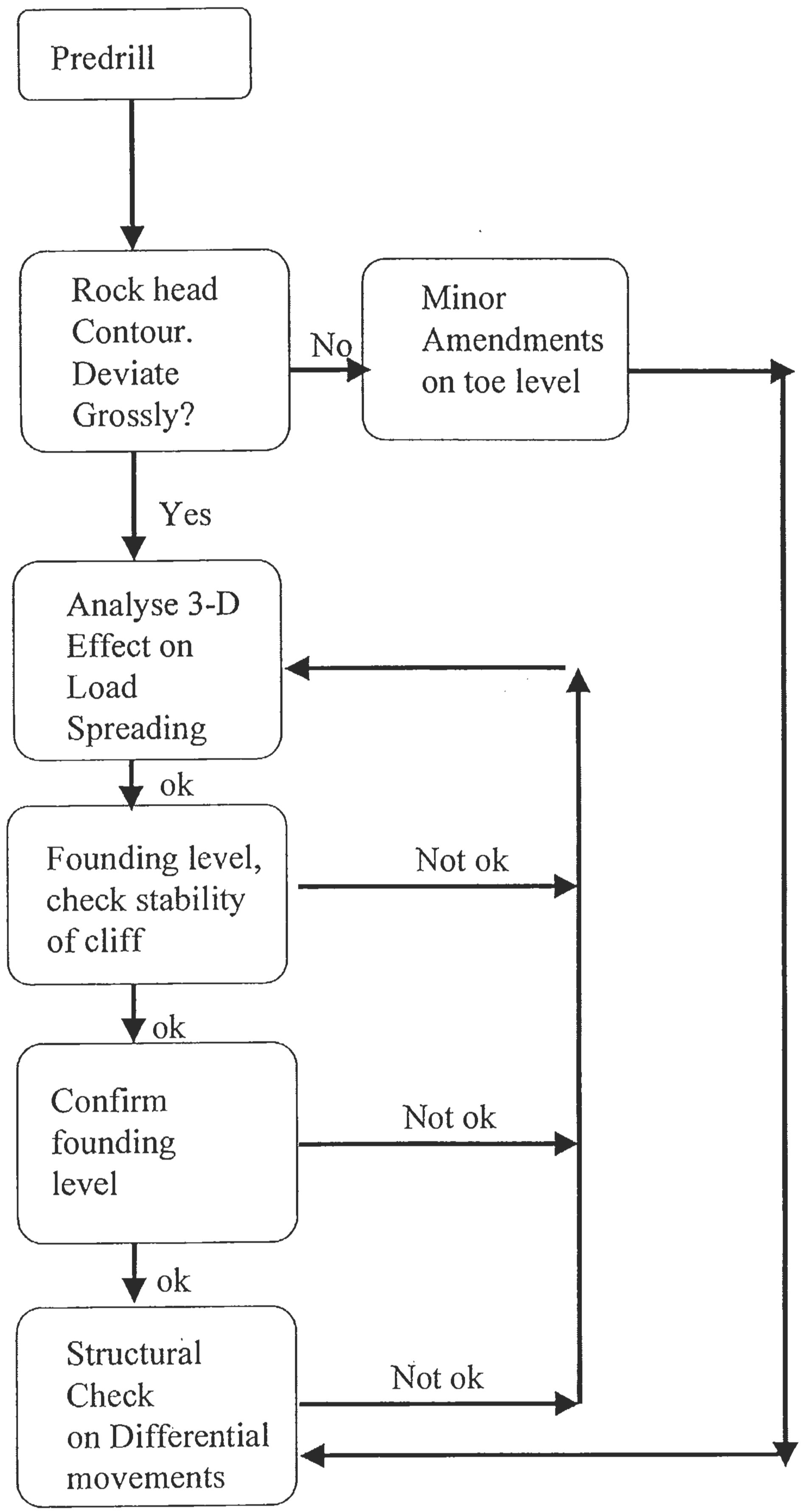
Geotechnical Control Office (1986) *Geology of Sha Tin. The Hong Kong Geological Survey Memoir No:1* Geotechnical Control Office, Hong Kong

Geotechnical Control Office (1987) *Geotechnical Area Studies Programme Report II. Central New Terrritories.* Geotechnical Control Office, Hong Kong

CM Tse, Paul Chan, Julian Tyson, Walter Chan & K K Yin (2002). "Geotechnical Aspects of the KCRC Ma On Shan Rail." *Geotechnical Aspects of Mass Transportation Systems.* Proc. 22nd HKIE Geotechnical Division Annual Seminar, Hong Kong Institution of Engineers, Hong Kong.

DISCLAIMER

The views expressed in this paper are those of the authors and do not necessarily represent the opinions of the KCRC.


Predrill
Rock head Contour. Deviate Grossly?
No
Minor Amendments on toe level
Yes
Analyse 3-D Effect on Load Spreading
ok
Founding level, check stability of cliff
Not ok
ok
Confirm founding level
Not ok
ok
Structural Check on Differential movements
Not ok

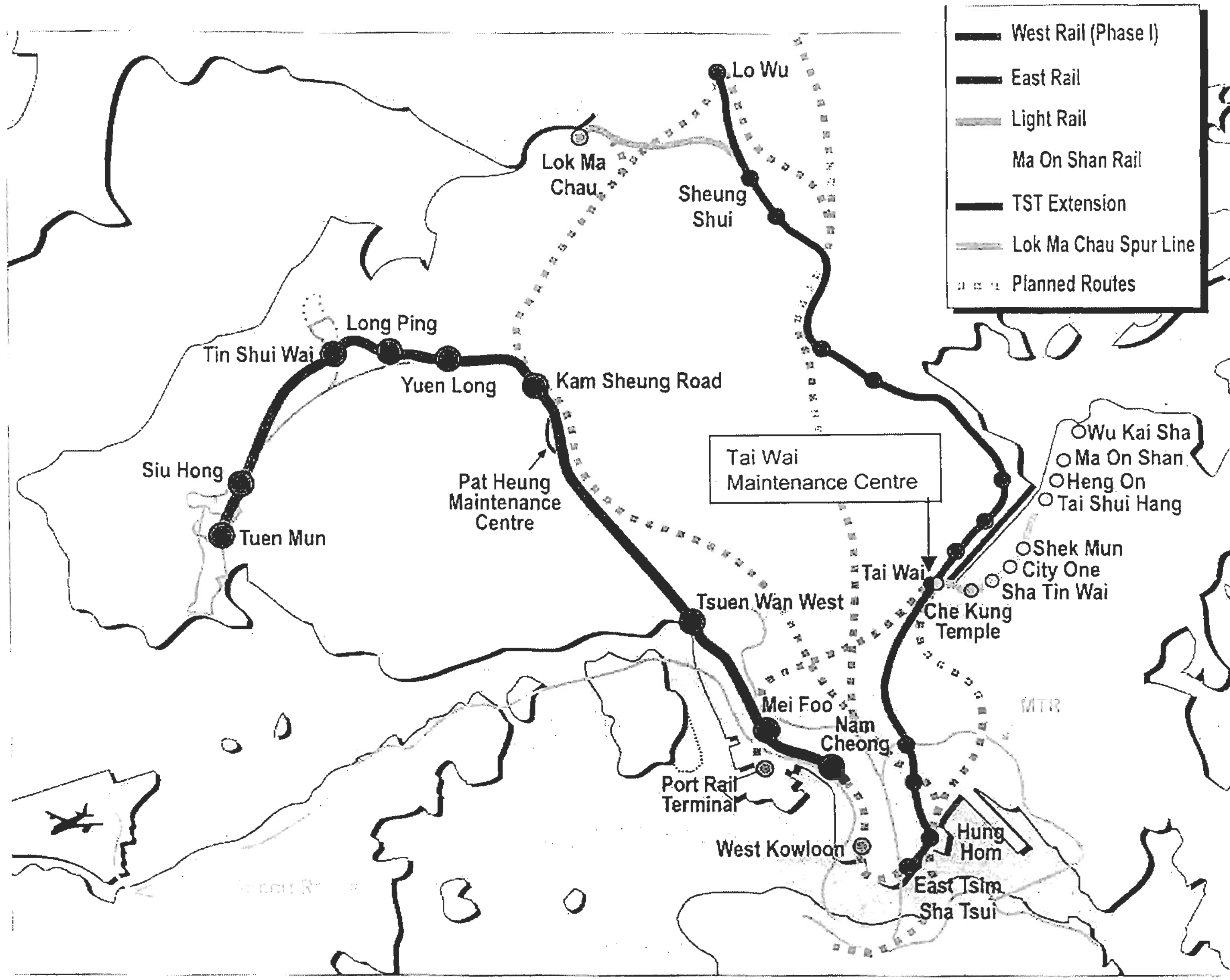


FIGURE 1 KCRC RAIL DEVELOPMENT

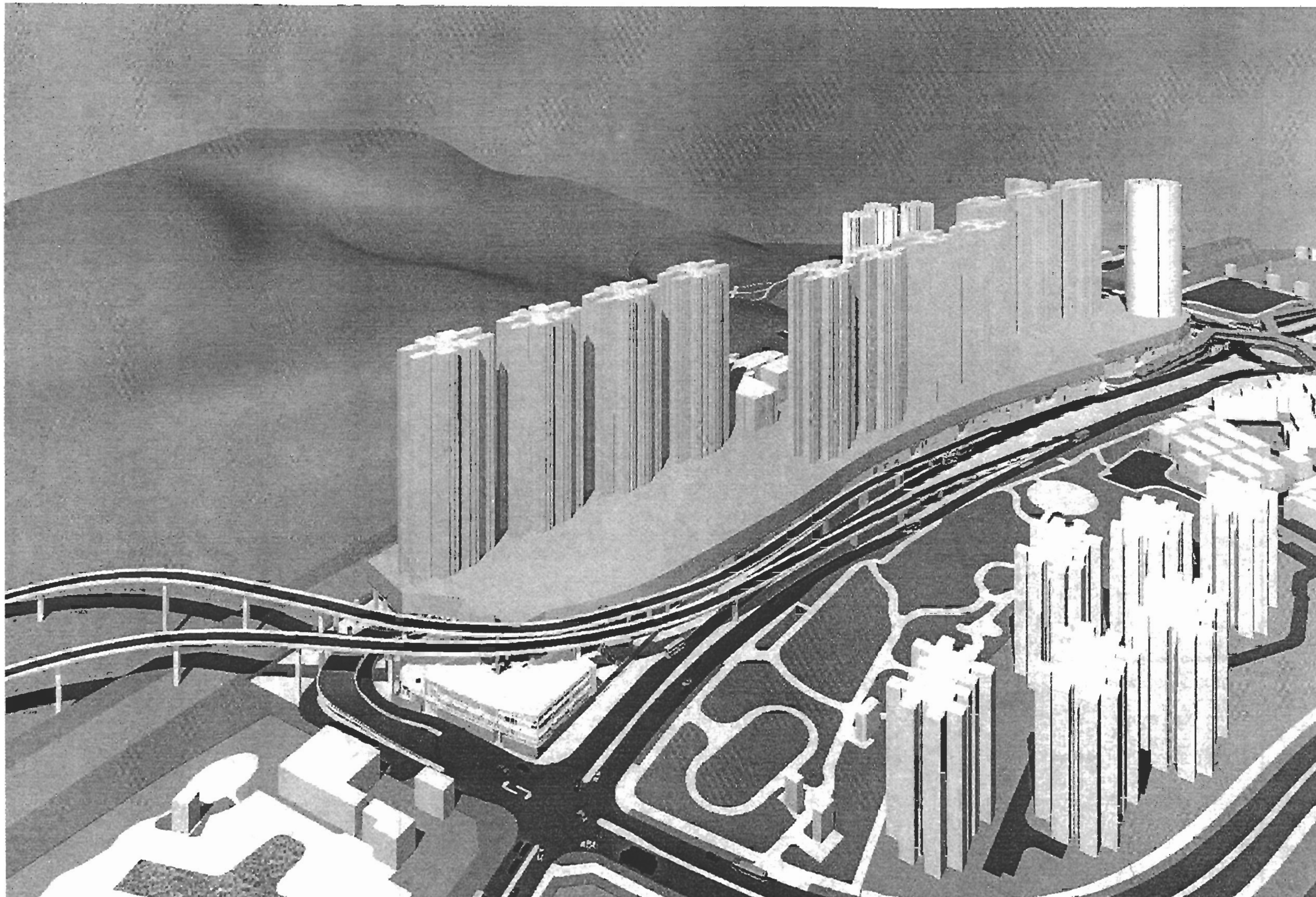


FIGURE 2 DEPOT WITH PROPERTY DEVELOPMENT ABOVE

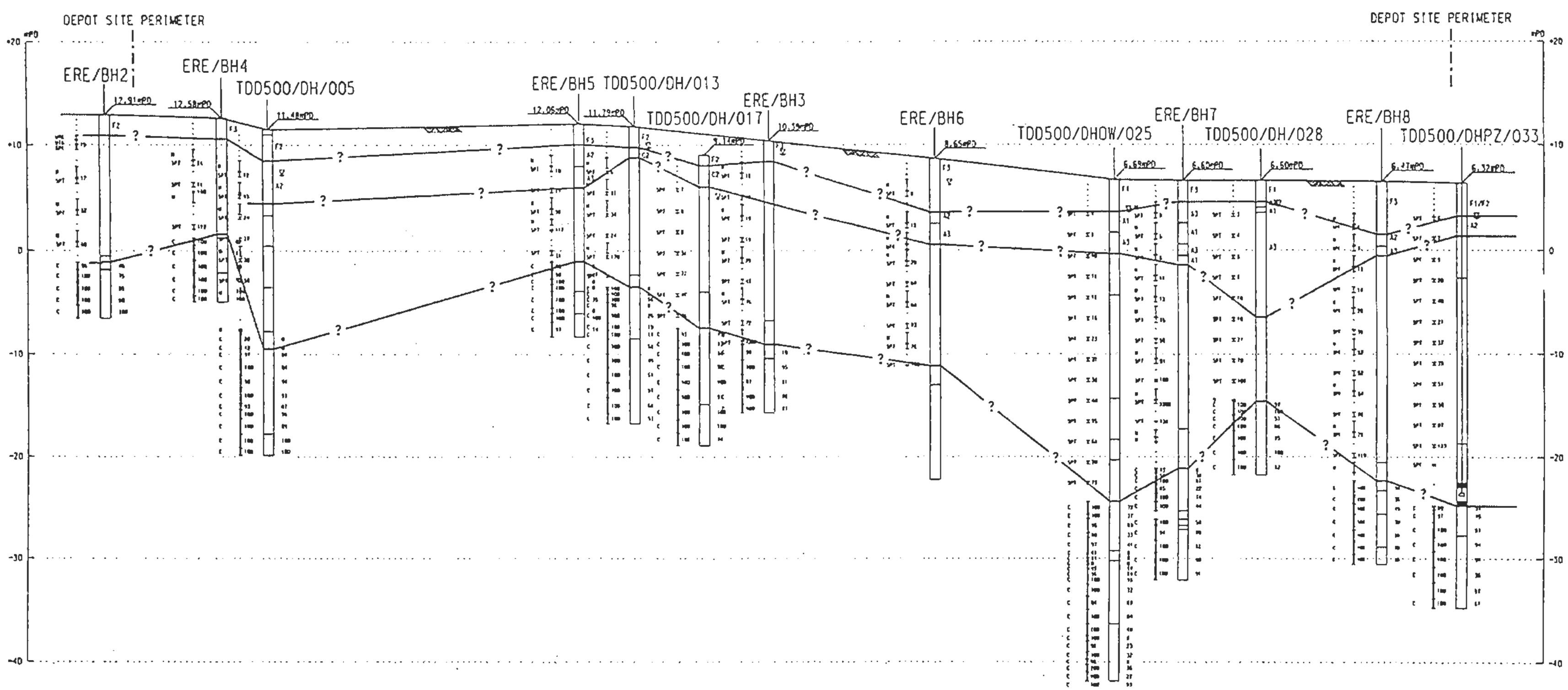
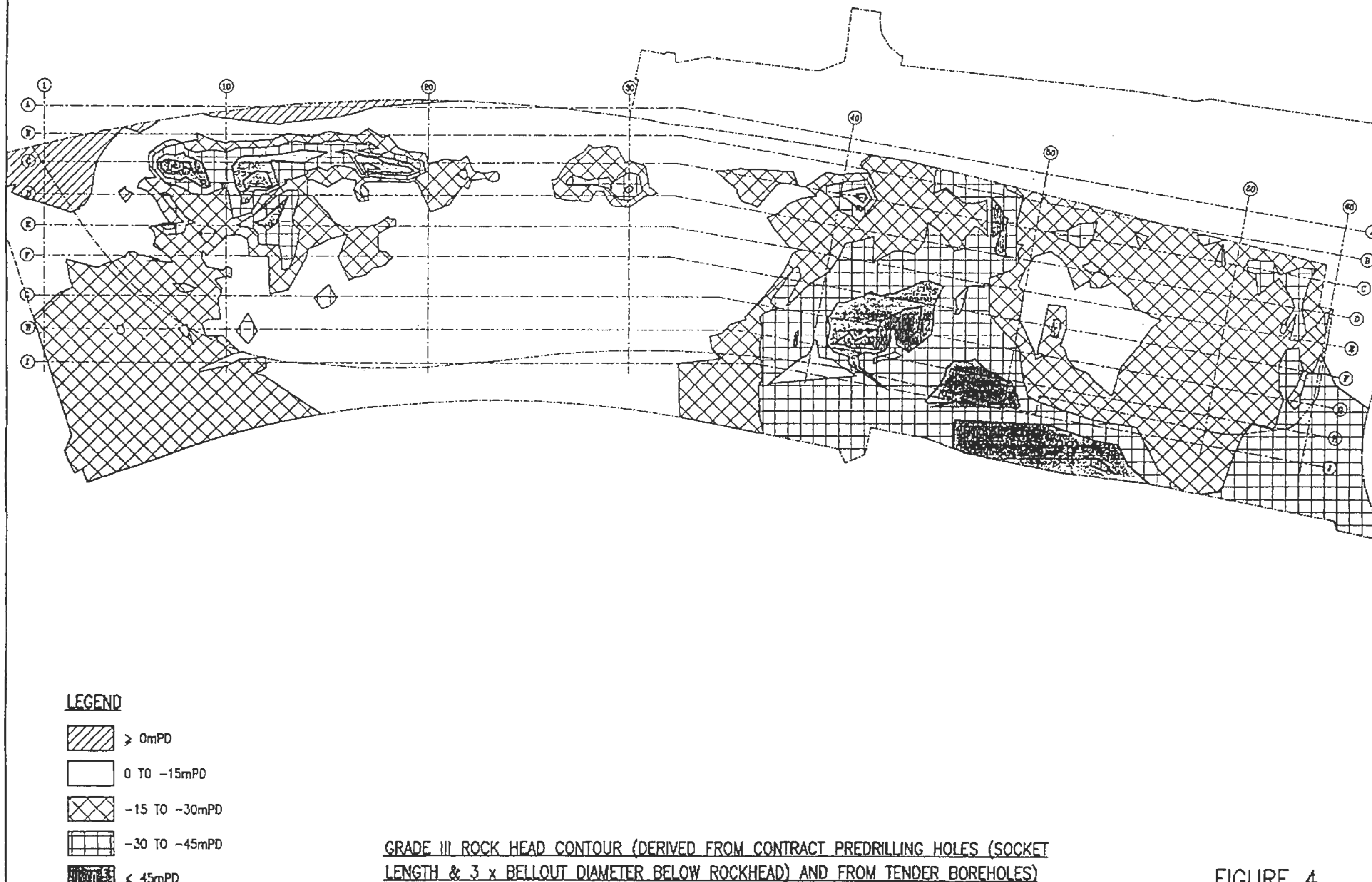
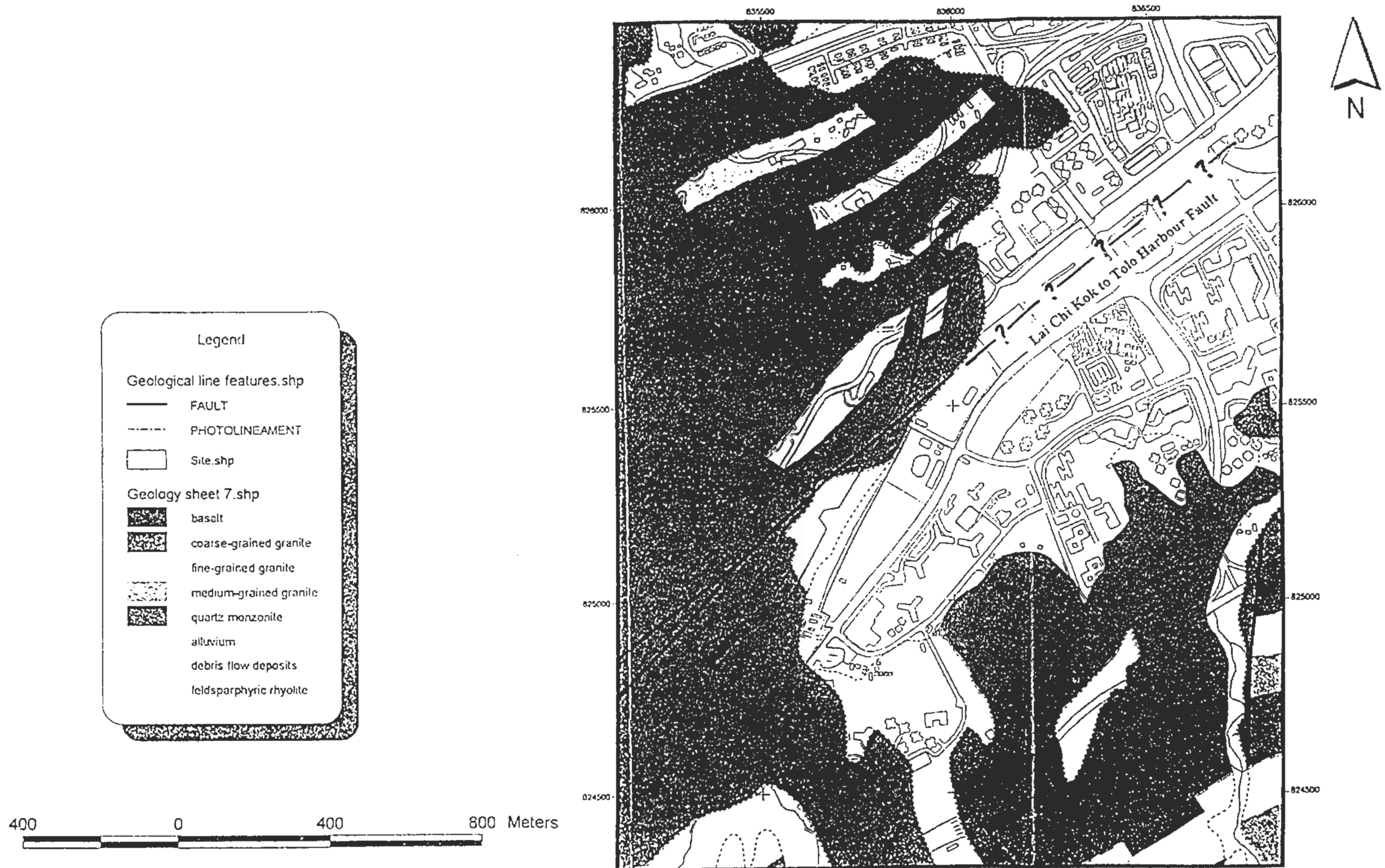


FIGURE 3
GROUND INVESTIGATION DRILLHOLE SECTION



GRADE III ROCK HEAD CONTOUR (DERIVED FROM CONTRACT PREDRILLING HOLES (SOCKET LENGTH & 3 x BELLOUT DIAMETER BELOW ROCKHEAD) AND FROM TENDER BOREHOLES)
(KCRC TAI WAI MAINTENANCE CENTRE)

FIGURE 4



ɪroduced from digital geological map sheet No 7 with permission of GEO/ Planning

FIGURE 5
TAI WAI MAINTENANCE
CENTER GEOLOGICAL MAP

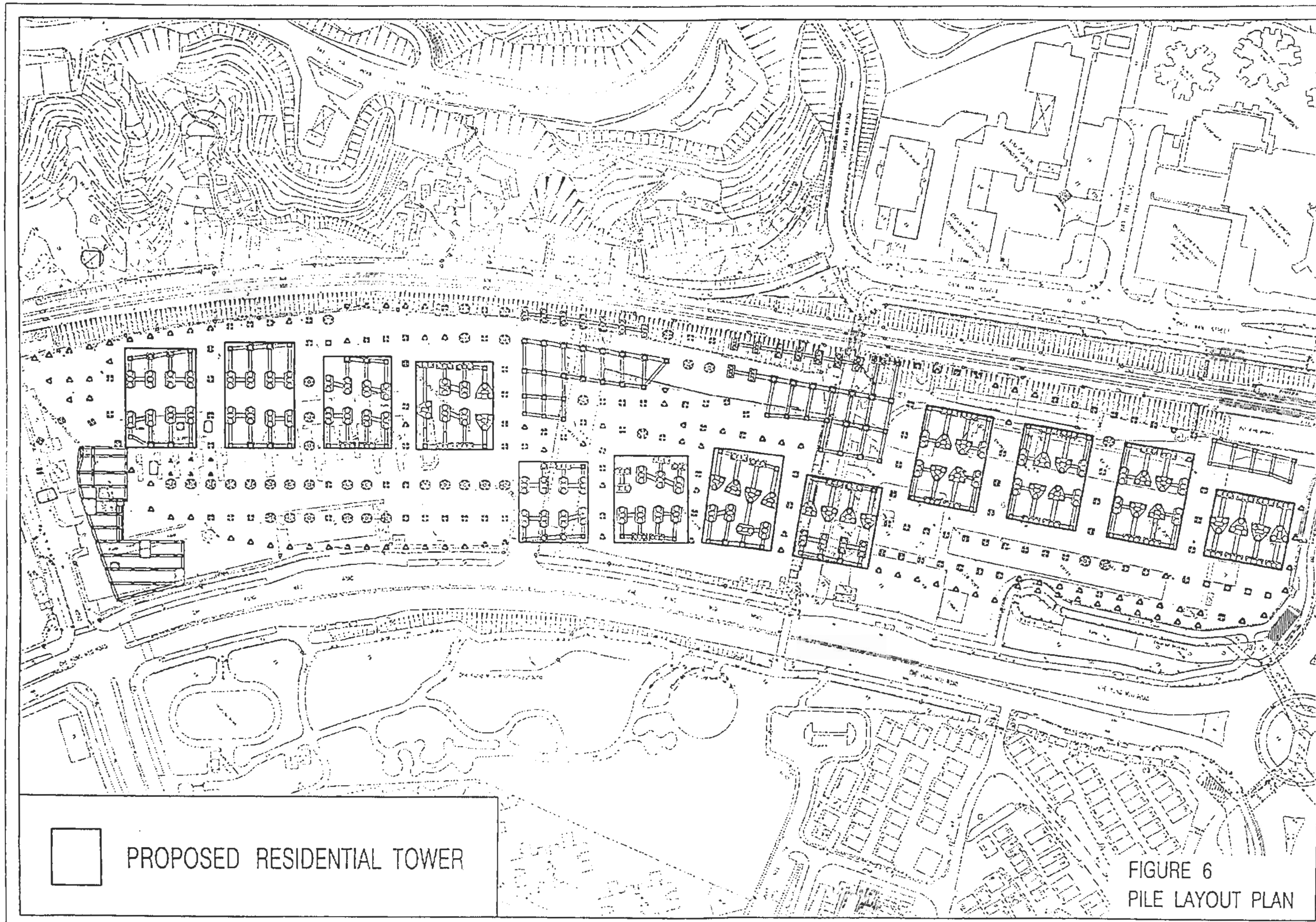


FIGURE 6
PILE LAYOUT PLAN

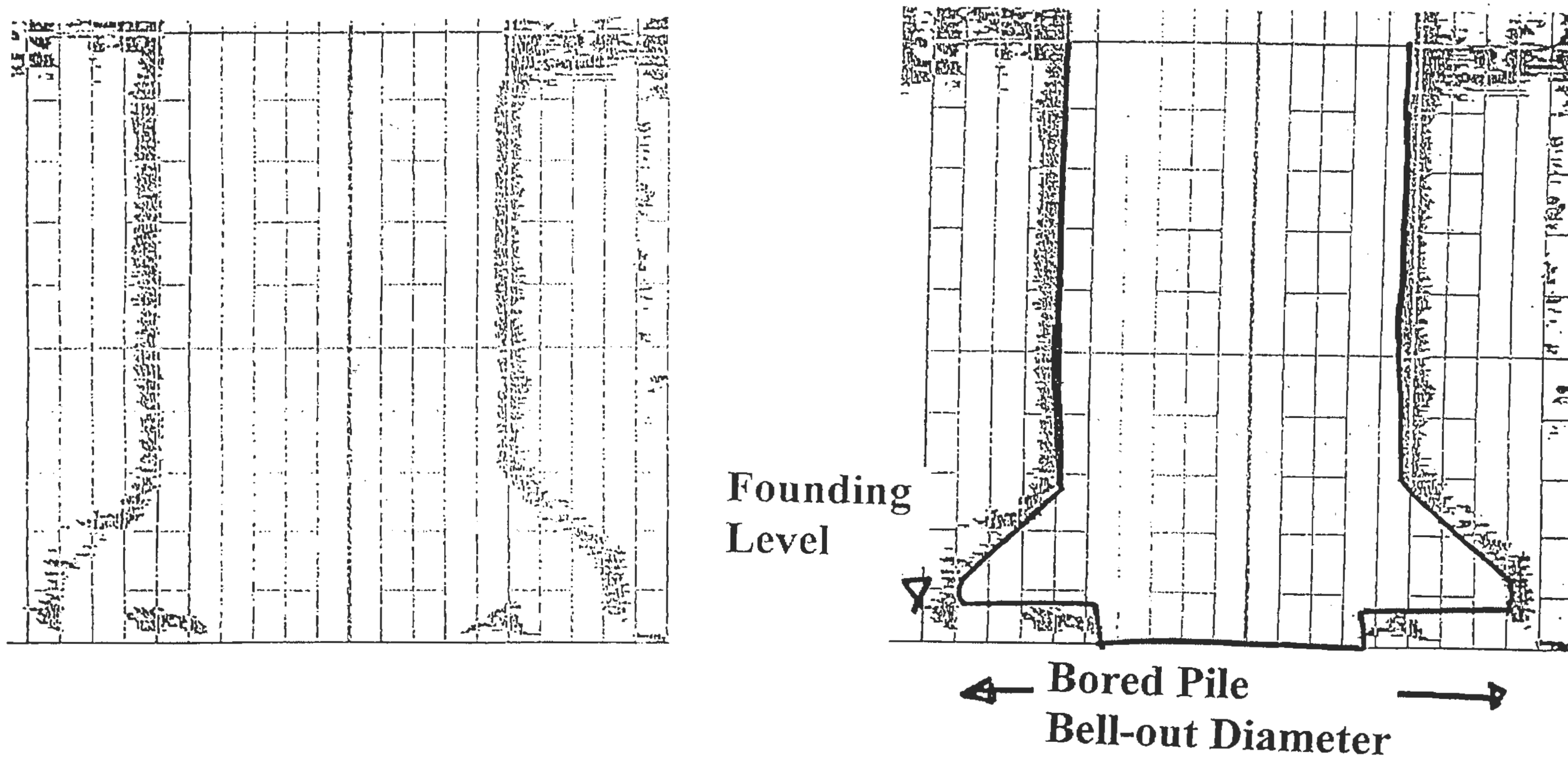


FIGURE 7

TYPICAL KODEN TEST PRINTOUT
BEFORE AND AFTER ASSESSMENT

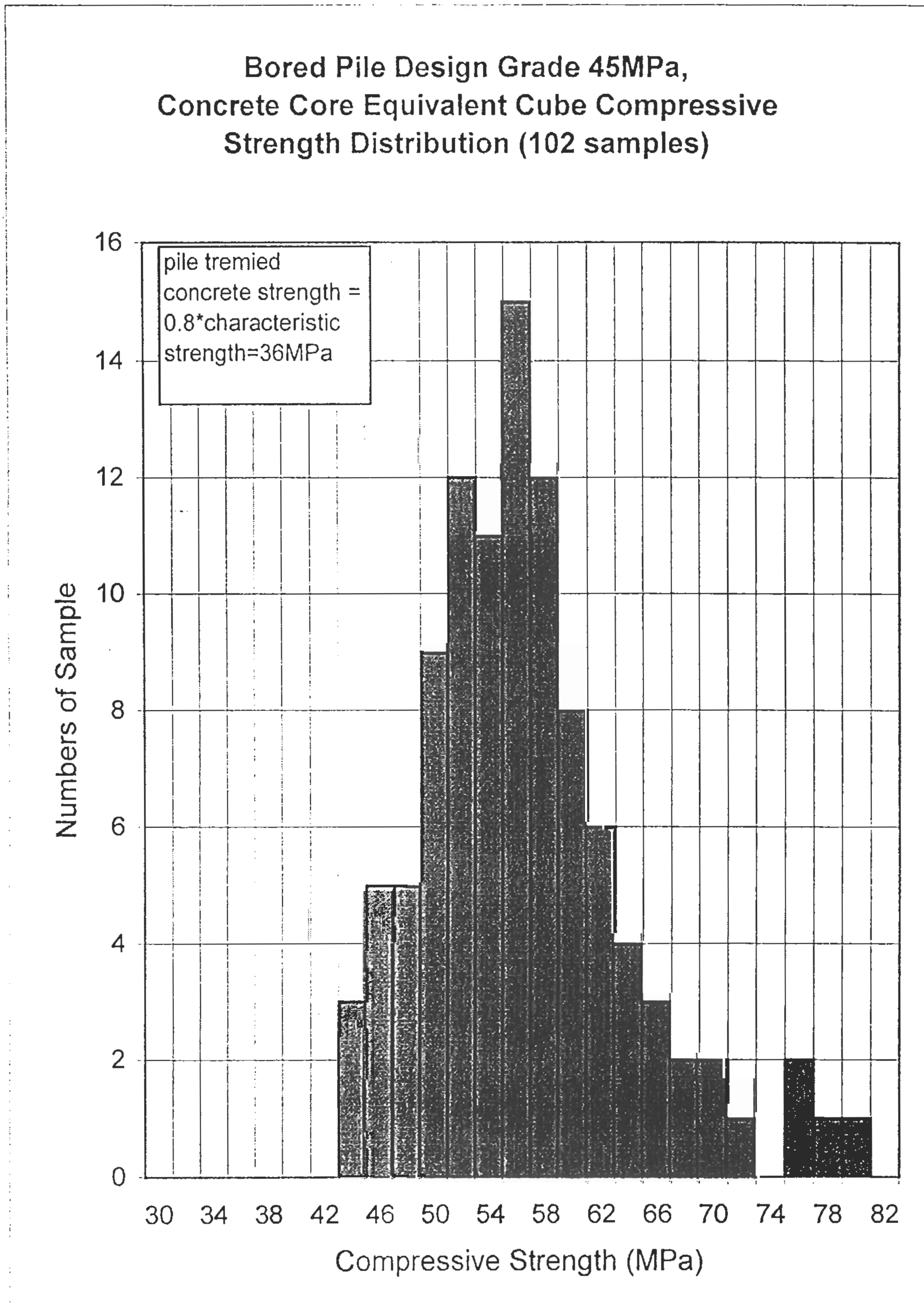


FIGURE 8 STATISTICAL DISTRIBUTION OF CONCRETE STRENGTH

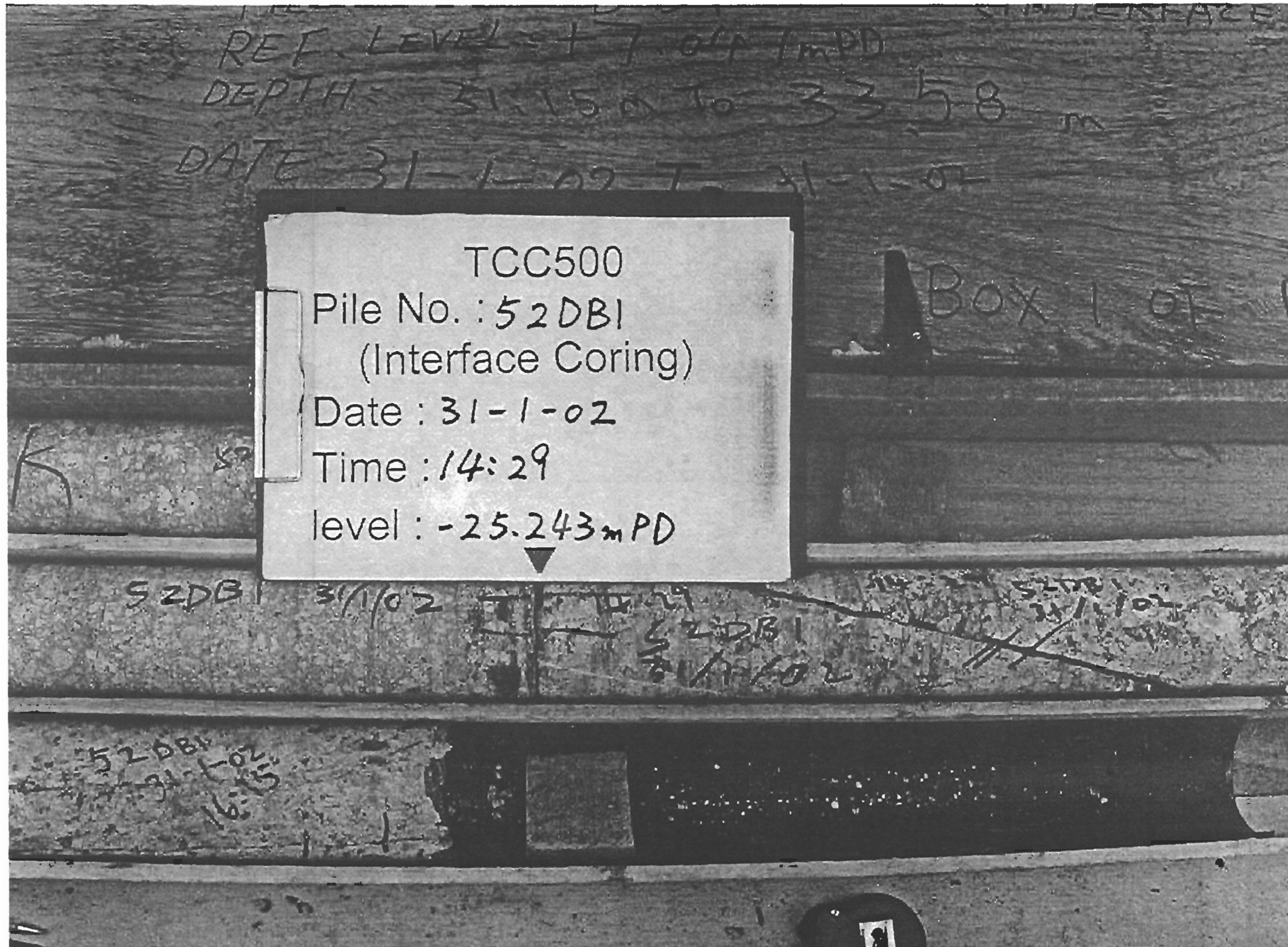


FIGURE 9 TYPICAL INTERFACE CORE

回灌水井在減輕九廣西鐵荃灣站
地表沈降中的應用

布賴恩 洛克裏夫 BACHY SOLETANCHE GROUP LTD.

安迪 皮克斯 GEOTECHNICAL CONSULTING GROUP (ASIA) LTD.

李紹威 GEOTECHNICAL CONSULTING GROUP (ASIA) LTD.

THE USE OF RECHARGE WELLS TO MITIGATE GROUND SETTLEMENT ON KCRC CONTRACT CC300 TSUEN WAN STATION AND APPROACH TUNNELS

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Bachy Soletanche Group Limited
Andy Pickles
Geotechnical Consulting Group (Asia) Limited
Siew Wei Lee
Geotechnical Consulting Group (Asia) Limited

摘要

九廣鐵路西鐵項目 CC300（荃灣及臨近隧道）採用“開挖－回填”的技術進行建造，其中還包括在新近開墾土地上安裝 2 公里的防護隔牆。在降水的過程之中，圍堰以外的一些區域出現了明顯的地下水壓力下降。通過對地下水壓力下降原因的分析，決定採用回灌水井的方法以最低限度的降低對地表及周圍結構的影響。在此，將對車站和隧道開挖以外區域的地下水壓力下降程度，採用不同灌漿方法對開挖防護隔牆底部進行支護的效果，及回灌水井對周圍地表的影響等問題進行討論。

THE USE OF RECHARGE WELLS TO MITIGATE GROUND SETTLEMENT ON KCRC CONTRACT CC300 TSUEN WAN STATION AND APPROACH TUNNELS

Brian A.W. Norcliffe[1]
Dr. Andy Pickles[2]
Dr. Siew Wei Lee[2]

ABSTRACT: The KCRC West Rail Contract CC300 "Tsuen Wan Station and Approach Tunnels" was constructed by cut and cover techniques, and involved the installation of 2 km of diaphragm walls in recently reclaimed land. During dewatering, significant piezometric drawdown was noted in certain areas outside the cofferdam. This piezometric drawdown was analysed, and recharge wells were designed to minimise the effects of the drawdown on the ground and adjacent structures. The magnitude of the piezometric drawdown outside the excavation for the Station and Tunnels is presented. The effectiveness of the different methods of grouting at the toe of the diaphragm wall surrounding the excavation and the behaviour of the ground with respect to recharge well operation are discussed.

INTRODUCTION

The Kowloon and Canton Railway Corporation (KCRC) has undertaken construction of a railway linking the Western area of the New Territories to Kowloon known as "West Rail". Contract CC300 for the Tsuen Wan West Station and associated Southern and Northern Approach Tunnels (SAT and NAT) comprises one of West Rail contracts. The railway station (400m in length) and approach tunnels (each 300m in length) were constructed using the cut and cover method. The north-eastern end of the site is shown in Figure 1.

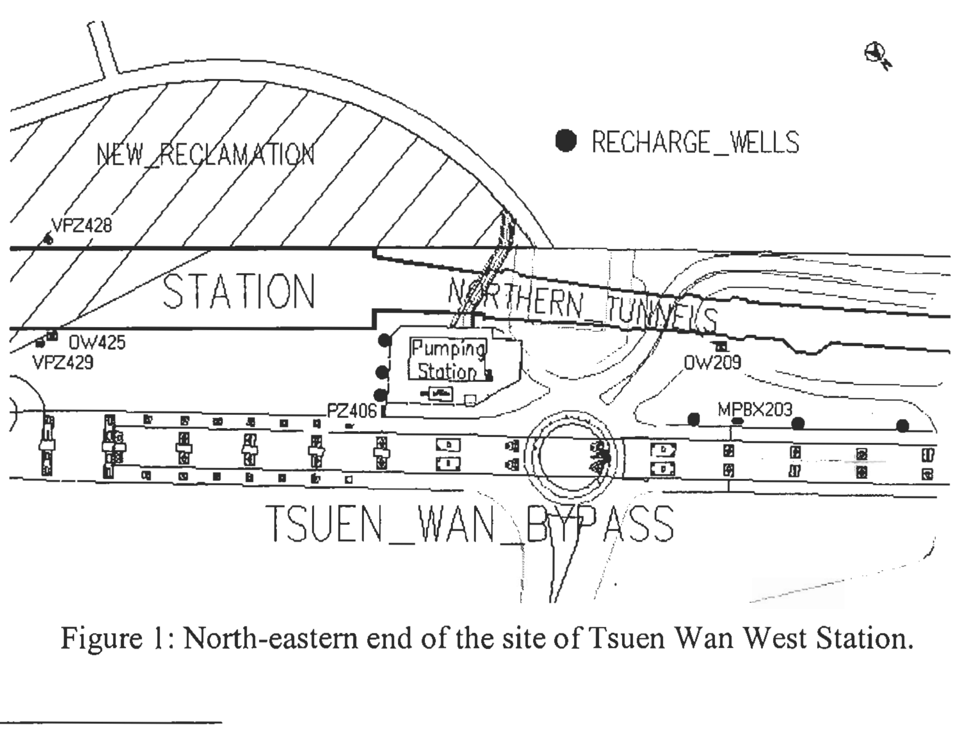

Figure 1: North-eastern end of the site of Tsuen Wan West Station.

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A number of existing structures are located adjacent to the CC300 site. These include the elevated 4 lane Tsuen Wan Bypass (TWB) viaduct, which runs parallel to the site at a distance of approximately 40m from the diaphragm wall, the Tsuen Wan Central Salt Water Pumping Station directly adjacent to the NAT and water and gas mains which cross the site. These structures had strict limitations imposed on allowable movement induced by construction of the Station and Tunnels. The locations of these structures are shown in Figure 1.

GROUND CONDITIONS

The Station and Approach Tunnels were constructed on reclaimed land. The north-eastern side of the site was reclaimed at various times up to 1990 and the south-western side of the site was reclaimed immediately prior to CC300 as part of the West Rail project. The extent of new and older reclamation is shown in Figure 1. The general features of the ground conditions with relevance to this paper are as follows:

- The new reclamation fill on the south-western side of the site is vibro-compacted marine sand. The soft marine clay that blankets the seabed in Hong Kong was removed prior to placement of the fill over the area of the new reclamation.
- The fill in the older reclamation comprises both general fill and public fill and is more variable than the newer marine sand fill. Although the majority of the fill is granular, more silty and clayey material is often encountered. In addition, a significant thickness of boulders or rock fill has been encountered, particularly in the area of former sea walls.
- The base of the fill is typically at a level between –8 and –10 mPD. The older reclamation fill is generally underlain by between approximately 2 to 5m of soft marine clay.
- The alluvium that underlies the fill or marine deposits is generally granular and has a thickness typically in the range of 2 to 5m. Alluvial clay has been encountered at some locations. The base of the alluvium is typically at a depth of –12 to –15 mPD.
- A variable thickness of completely decomposed Granodiorite underlies the alluvium. The level of the rock head at the site is variable, being encountered between approximately –12 mPD and –40 mPD. The rock underlying the site is highly fractured in some areas.

THE GROUNDWATER CUT-OFF

Prior to construction, groundwater levels within the new marine sand fill were tidal and typically ranged between +0.5 and +2.5 mPD. The water table within the older reclamation fill was not tidal but was more strongly influenced by seasonal effects and typically varied between +2 and +4 mPD. The cofferdam constructed around the Station and Tunnels comprised a diaphragm wall with a thickness varying between 800 and 1500 mm. During excavation the diaphragm walls were supported by multi-layer propping. Pumping wells were installed within the excavation to drawdown water levels to one metre below the founding level of the Station and Tunnels (between –13 and –17 mPD). Three different methods of grouting were adopted to improve the ground conditions at the toe of the diaphragm walls. All three methods were carried out through reservation tubes that were attached to the diaphragm wall reinforcement cages. The three grouting methods are summarised as follows:

a) Contact grouting was used to both improve the bearing capacity of the rock and to locally reduce the permeability of the rock where the founding material was Grade

III rock or better. Contact grouting involved cement/bentonite grouting to a depth of approximately 1m below the toe of the wall.

b) Rock grouting was used to provide a better groundwater cut-off beneath the landward Station diaphragm walls that are closer to the TWB than the Approach Tunnels. Rock grouting involved cement/bentonite and microfine cement grouting to a depth of 5m below the toe of the wall.

c) TAM (Tube à Manchette) grouting was performed underneath the diaphragm wall where the founding material was Grade IV weathered rock.

As there were no sensitive structures on the seaward side of the site and also as a result of the obvious potential for rapid ground water recharge from the sea, there was no concern over the groundwater drawdown on the seaward side of the site. No grouting was therefore required on this side of the station.

Following completion of the diaphragm walls, a series of pumping tests were performed to test the effectiveness of the groundwater cut-off. The Station and Approach Tunnels were divided into a number of discrete compartments by temporary bentonite cement cut-off walls. This enabled the pumping tests to be carried out as completed sections of diaphragm wall became available. As indicated by the pumping tests and as subsequently proven during the excavation, no inflows were recorded across the diaphragm walls. The only inflows into the excavation were from beneath the diaphragm walls through discontinuities in the rock.

Instrumentation

A number of observation wells (OW) were installed to monitor the groundwater level in the fill. The majority of these instruments were installed in three rows at a spacing of approximately 50m. Two of the rows were within 2 to 5m of the outer face of the seaward and landward diaphragm walls, and the third row was located along the Tsuen Wan Bypass.

Two rows of vibrating wire piezometers (VPZ) were installed within approximately 5m of the outside face of the seaward and landward diaphragm walls, with their tips located approximately 0.3m above the rock head. A number of piezometers were also installed at rock head adjacent to the Tsuen Wan Bypass.

Various instruments to monitor ground and structure movements were installed, including ground surface and structure movement points, Multiple Point Borehole Extensometers (MPBX) along the alignment of the Tsuen Wan Bypass and inclinometers in the diaphragm wall and adjacent to the Tsuen Wan Bypass. The locations of the instruments specifically referred to in the paper are shown in Figure 1.

Response of Ground Water Level in Fill to Dewatering

As a result of the relatively high permeability of the marine sand fill and the proximity of the rubble sea wall, the water level in the new reclamation fill was tidal. As would be expected, the OWs on the seaward side of the site in the new reclamation fill showed no response to the dewatering required for construction of the Station and Tunnels.

The majority of the OWs on the landward side of the site indicated that the ground water level in the fill was influenced by the dewatering. The OWs adjacent to the Station and

NAT diaphragm walls indicate a relatively slow reduction in the groundwater level as a result of dewatering for the Station and NAT. The response at OW425 shown in Figure 2 is typical. Dewatering at the north-eastern end of the station commenced in September 2000 and the ground water level reduces slowly thereafter to a level of 0.5 mPD by March 2001, which is approximately 1m below the mean sea level. The level rises again in June in response to the heavy rainfall, which occurred in June and July, although this rise was also possibly affected by a reduction in the extent of dewatering in the Station.

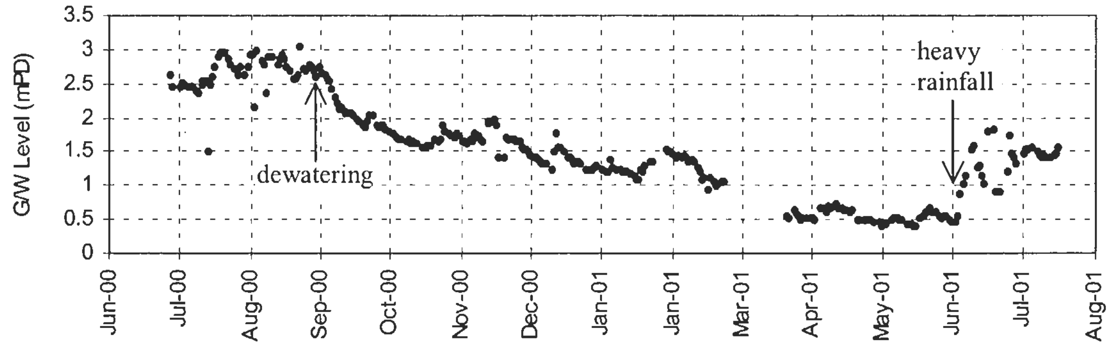

Figure 2: Groundwater data at OW 425.

The drawdown adjacent to the diaphragm wall was typically less than 1m. The maximum drawdown adjacent to the diaphragm wall occurred at OW209, where the groundwater level was reduced to –1.5 mPD.

The excavation and dewatering appears to have had little influence on the groundwater level adjacent to the TWB where the groundwater appears to be influenced only by seasonal/rainfall events.

RESPONSE OF PIEZOMETRIC PRESSURE AT ROCK HEAD TO DEWATERING

Prior to construction of the diaphragm walls the majority of the VPZs at rock head level show some pressure fluctuation as a result of tidal effects. The magnitude of the response is a function of the distance of the piezometer from the seawall, the state of the hydraulic connection through the ground between the sea and the piezometer tip and the presence of the diaphragm walls. In order to illustrate this it is worth comparing the data recorded at VPZ428 and VPZ429 which are situated close to the outside face of the diaphragm walls but on opposite sides of the Station. The data obtained from VPZ428 and VPZ429 is shown on Figure 3 and the following points can be seen:

- Prior to construction of the diaphragm wall the daily pressure response at VPZ428 varies by approximately 2.0 m whereas the daily pressure response at VPZ429 varies by only approximately 1.2 m. The smaller daily pressure fluctuation at VPZ429 is because this piezometer is located approximately 50 m further from the sea than VPZ428.
- The diaphragm walls in this area were completed around the beginning of August and it can be seen that the daily pressure variation at VPZ429 after the beginning of August decreases to approximately 0.3m, demonstrating the effectiveness of the diaphragm walls at isolating the tidal response. The daily pressure variation at VPZ428 remains unchanged although there is a response to both dewatering and also a seasonal/rainfall variation.

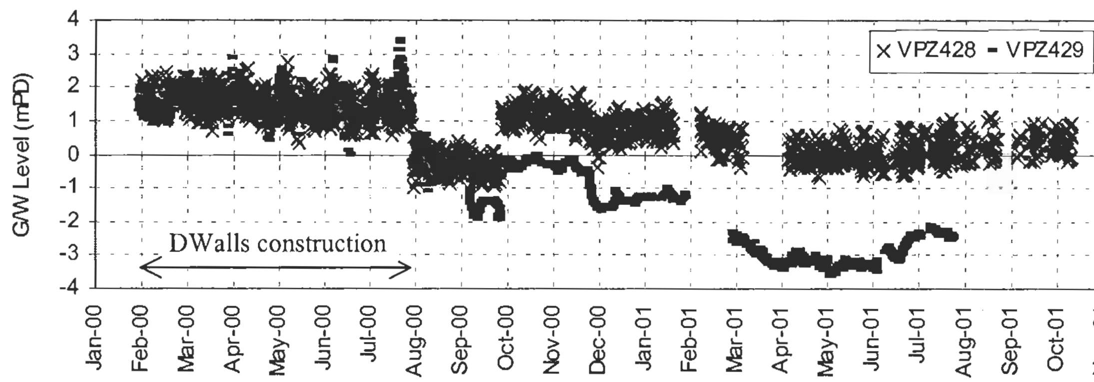

Figure 3: Groundwater data at VPZ 428 and 429.

As demonstrated in Figure 3, the piezometric pressure at rock head has responded to the dewatering carried out for construction of the Station and Tunnels. The magnitude of the reduction is a function of a number of factors including the level of drawdown and pumping rate from within the excavation, the localised ground conditions and the extent and effectiveness of the contact and rock grouting below the toe of the diaphragm wall. The maximum drawdown recorded by the rock head piezometers adjacent to both the seaward and landward diaphragm walls are summarised in Figure 4.

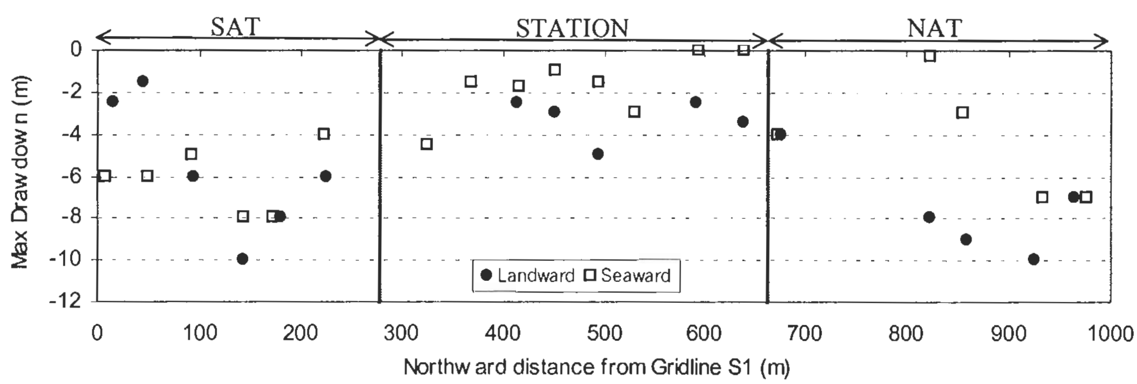

Figure 4: Maximum drawdown recorded by landward and seaward VPZs.

The following features relating to drawdown of water pressure adjacent to the diaphragm walls can be made based on the results shown in Figure 4.

- Although there is a large variation in the results, the average drawdown adjacent to the SAT, Station and NAT is to a level of approximately –5, -0.5 and –4 mPD respectively. This represents a reduction of 6.5, 2 and 5.5m below the mean sea level.
- The average drawdown at rock head level is less at the Station than at the adjacent NAT and SAT. This gives an indication of the benefit of the rock grouting carried out beneath the landward station walls. Groundwater recharge was adopted at the north-eastern end of the station box adjacent to the Tsuen Wan pumping station and this has had an influence on the results in this area.
- Despite no rock grouting on the seaward side of the station the drawdown is relatively small due to the rapid recharge of groundwater pressure from the sea, indicating a relatively high permeability path in this area of the site.

- The contact grouting beneath the NAT appears to have had only a relatively small effect on the magnitude of the drawdown when compared with the results for the SAT where no contact grouting was performed.

A number of piezometers were also installed at rock head level adjacent to the TWB, which is approximately 40m from the Station and Tunnels. The drawdown adjacent to the TWB was typically in range of 30% to 60% of the drawdown adjacent to the wall with an average value of 40%.

Requirement for Recharge Wells

During the pumping test in the NAT, despite the relatively low flow rate recorded into the tunnel box, the VPZs adjacent to the diaphragm walls indicated a significant drawdown of up to 8m at rock head. A negligible drawdown was recorded by the OWs in the reclamation fill. In this area of older reclamation the clayey marine deposit at the base of the fill acts as an aquitard, resulting in the CDG and alluvium exhibiting the behaviour of a confined aquifer.

An additional piezometer was installed at rock head level adjacent to the TWB and this indicated the drawdown due to dewatering was migrating towards the TWB. Multiple Point Borehole Extensometers (MPBXs) adjacent to the TWB also indicated that ground settlement associated with the pumping test was occurring in the various different strata below the TWB.

The response of the VPZs during the pumping test raised the concern that during the long term dewatering required for construction of the NAT the piezometric drawdown would migrate towards the TWB, inducing ground settlement and negative skin friction on the supporting piles. A similar concern was also noted in the area of the Tsuen Wan Salt Water Pumping Station.

Recharge Well Design

The piezometer and VPZ data obtained during the pumping test were used to calibrate a finite difference model for groundwater flow. Recharge wells were then modelled to determine the most appropriate design and location in order to prevent drawdown adjacent to the TWB and Pumping Station. The design also had to consider the risk of piping within the tunnel excavation if the recharge head was too great.

The re-charge system consisted of 219 mm diameter recharge wells, which were installed to a minimum depth of 5m below the top of the CDG, or to the top of the Grade III rock head where rock head was less than 5m below the top of the CDG. A 50 mm pipe was installed in the well hole. The pipe was perforated from a level of 1m below the top of the CDG and wrapped with two layers of nylon mesh (2 mm pores). The well hole was then backfilled with gravel to 1m below the top of the CDG. A 1m thick sand filter was placed on top of the gravel and the remainder of the hole was filled with cement/bentonite grout.

Each recharge well was connected to a header pipe that was supplied by a water silo, filled to a constant height of 5m above ground level. The silo base was founded at about +4.5 mPD. If, for example, groundwater levels were drawn down to -2.5 mPD then there was 12m of excess pressure head to drive the system. Once the system was operating flow metres showed that the recharge wells were essentially self-regulating. As expected, when greater pressure head differences occurred, greater volumes were recorded by the flow meters. When piezometers showed that the piezometric pressures had returned to their original levels of around +2 mPD very little flow was recorded.

The locations of the three recharge wells adjacent to the Pumping Station and the three recharge wells adjacent to the TWB are shown in Figure 1. Very little maintenance work was required for the recharge wells adjacent to the TWB. The rate of water inflow at the recharge wells adjacent to the Pumping Station decreased with time and it appeared that the wells were becoming blocked. Although the reason for this blockage was not determined it was found that high pressure water jetting removed the blockage and increased the efficiency of the recharge wells.

PERFORMANCE / ASSESSMENT OF THE RECHARGE WELLS

In the area of the Pumping Station the effectiveness of the recharge was monitored by piezometers installed at rock head close to the Pumping Station. The water pressure recorded at piezometer PZ406 is shown in Figure 5. The ground water recharge commenced in this area in November 2000. Prior to November the piezometric pressure had reduced by approximately 20 kPa (2m head).

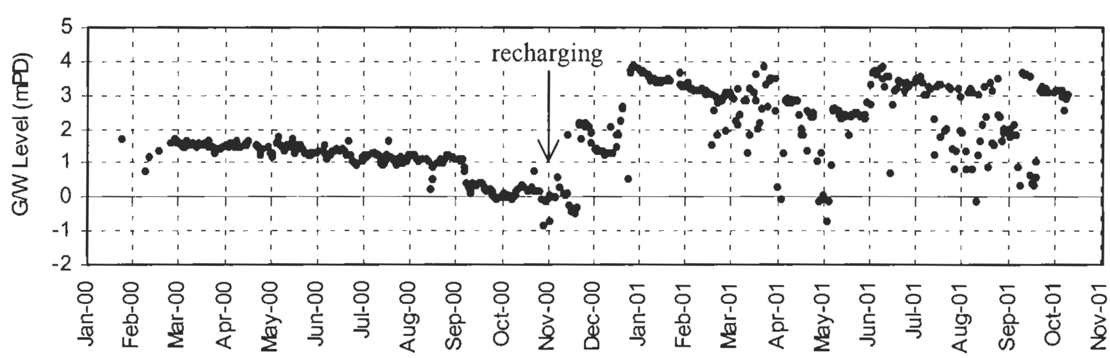

Figure 5: Piezometric data at PZ406.

On commencement of the recharging an immediate increase in the water pressure at rock head was observed at all of the piezometers in the area of the Pumping Station. The rate of recharge was varied and the associated response of the piezometers can be seen in Figure 5. Although not presented in the paper it is noted that operation of the recharge wells immediately reduced the rate of ground settlement in this area and movement of the Pumping Station and associated pipe-work remained within the specified limits.

The effectiveness of the ground water recharge adjacent to the TWB was monitored by piezometers at rock head and by extensometers installed adjacent to the TWB. The results obtained at extensometer MPBX203 are presented in Figure 6. Ground water recharge adjacent to the TWB commenced in April 2001. Prior to this time the ground adjacent to the TWB was settling at a rate of approximately 2mm per month. On commencement of the recharging the ground settlement immediately stopped.

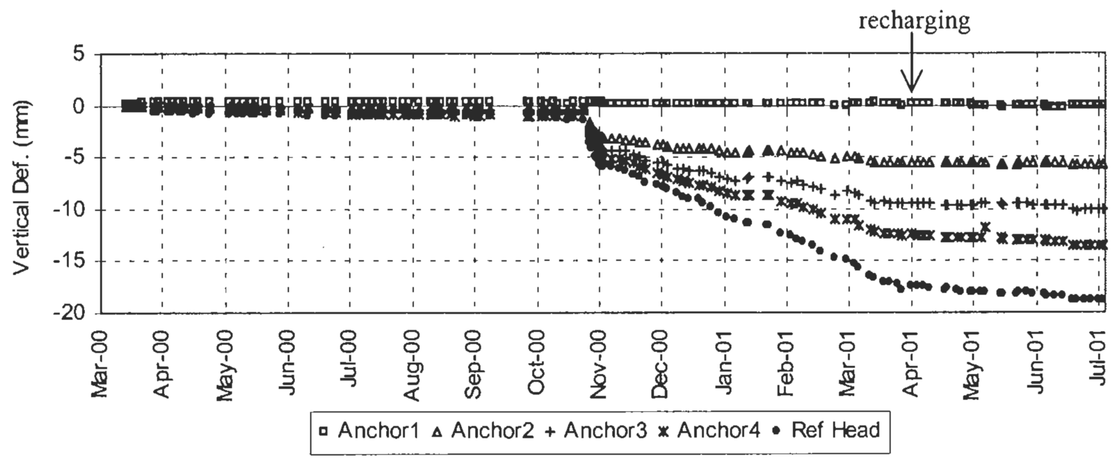

Figure 6: Ground settlement recorded by MPBX203.

SUMMARY

Construction of the Tsuen Wan West Station and Approach Tunnels required dewatering to a depth of approximately 15m below the typical ground water table. The excavation and dewatering was carried out within a cofferdam constructed using diaphragm walls generally taken down to rock head. Contact grouting was carried out at the toe of the wall in the NAT and Station, and in the area of the Station, additional rock grouting was carried out to a depth of 5m below the toe of the wall.

At locations where only contact grouting was adopted an average piezometric reduction of approximately 70 kPa was measured at rock head close to the wall. At locations where rock grouting was also carried out the average piezometric pressure reduction was limited to approximately 20 kPa. The piezometric pressure reduction at rock head at a distance of approximately 40m from the Station and Tunnel construction was found to be approximately 40% of the value measured at rock head close to the wall

The marine clay below the older reclamation on the north-eastern side of the site acted as an aquitard resulting in the alluvium and CDG below the fill acting as a confined aquifer. This in turn resulted in the water pressure reduction due to dewatering migrating further from the Station and Tunnels and small ground settlements were recorded in the area of the TWB and Pumping Station. A limited number of recharge wells were therefore installed and these successfully restored the water pressure and eliminated further ground settlement.

ACKNOWLEDGEMENTS

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承嵌於充填大理岩的挖孔樁的設計與建築

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THE DESIGN AND CONSTRUCTION OF BORED PILE SOCKETED IN INFILLED MARBLE

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撮要

九廣鐵路西鐵的部份路線將橫跨新界西北常遇的岩溶環境。在擬建的朗平站東面，高架橋的地基爲能夠建座於比較少溶洞的大理岩石上，必須穿過一連串的充填溶洞，直達地底一百米的深度。承建商提出一個替代的設計方案，是利用岩石體產生的嵌岩孔摩擦力再加上對充填溶洞的前期和後期處理。設計參數由一支爲場地特設及足尺的試樁得出。此文章會就這替代設計的概念及試樁的細節作出討論及解釋，亦會加以強調就採用這設計方案時對複雜地質的充份理解及組織一個合適的地質設計模型的重要性。

The Design and Construction of Bored Pile Socketed in Infilled Marble

Thomas Domanski[1], D.M. Lee[2], Hugh Boyd[3] & Francis Leung[4]

ABSTRACT: Part of the KCRC West Rail passes through the northwest New Territories where karstic environment is encountered. Just to the east of the proposed Long Ping Station, the foundation of the viaduct has to penetrate a series of infilled cavities, down to a depth of around 100m below ground level in order to found on marble rock that is relatively free from cavities. An alternative was proposed by the contractor, making use of the rock socket friction from the rock mass with pre and post treatment to the infilled cavities. The design parameters were acquired through a full-scale site-specific trial pile. In this paper, the concept of the alternative design is discussed and details of the trial pile are presented and interpreted. The importance of a good understanding of the complex geology and forming an appropriate geological model for design is emphasised when adopting this design approach.

INTRODUCTION

The proposed KCRC West Rail is a 30km long railway linking the northwest New Territories to the western side of Kowloon, Hong Kong. One of the construction contracts, CC201, consists of a viaduct that passes through the populated towns of Yuen Long, Long Ping and Tin Shui Wai in the NW New Territories. Ove Arup & Partners Hong Kong Limited (Arup) was the Engineer of the project and the viaduct was contracted by Maeda Chun Wo JV (MCWJV), to an alternative design by Robert Beniam & Associates (RBA). Part of the viaduct was constructed within the Scheduled Area No. 2, which is an area identified by the Building Authority of Hong Kong to contain marble rock that usually has a karstic upper surface with solution features. An increasing number of sites with large cavities have been found in the marble rock as more developments, either high-rise or long-span structures, are being designed to be supported by deep foundations.

In a particular stretch adjacent to the proposed Long Ping Station, the ground conditions were found to be very complex and significantly different from that of Yuen Long Station about 1km away. The marble rock was found to have a series of infilled layers of random thickness. No significant voids were found. Under this type of situation, the conventional end bearing piling design calls for bored piles to penetrate through all the infilled layers until continuous sound rock is found. For this particular area, this would require piles of length well over 60m and often longer than 100m.

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Bauer Hong Kong Limited, a subcontractor to MCWJV, based on their experience in the karstic areas in Kuala Lumpur and elsewhere, prepared an alternative jointly with Arup, RBA and KCRC, using socketed shaft friction in the infilled marble rock and demonstrated the effectiveness using a full-scale pile loading test. The design of the alternative, test details, results and its application are discussed in this paper.

SITE SETTING AND SITE GEOLOGY

The site is located within the viaduct section between Long Ping Station and Yuen Long Station and immediately adjacent to Long Ping Station (Figure 1). At both gridlines 418 and 419, the piles are designed to support the viaduct using a portal substructure which spans over the existing nullah.

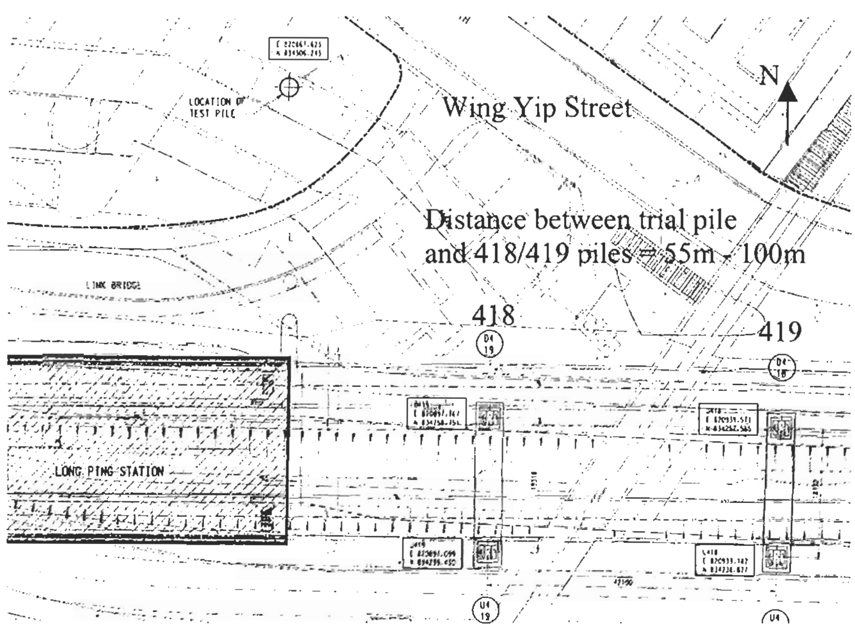

Figure 1 - Site Plan

The Yuen Long area has been designated as the Scheduled Area No. 2 after the discovery of marble and complex solid geology in the late seventies. The general geology of the area has been described by Frost (1992).

This section of the viaduct crosses a strong, negative anomaly that is seen on the gravity map (Arup, 1998). This anomaly, appears towards the middle of this viaduct section, corresponds with a very deep zone of deep weathering in which drillholes have been terminated at 150m depth without encountering quality rock. This deep weathering corresponds to the position of the granodiorite that was mapped by the Geological Survey as a continuation of the body met at the Yuen Long Industrial Estate (ie north of Long Ping station).

The rock beneath Long Ping station is marble interbedded with metasiltstone forming a transitional zone between the underlying Yuen Long Formation and the Lok Ma Chau Formation above. This block of rock is separated from the weathered granodiorite by a fault passing through the east side of gridlines 418 (refer to drillholes TS200/DHPZ/202 and 203 from the Technical Study stage). Although other faults have been shown passing through Long Ping Station on previous reports and maps, it is by no means certain that they exist.

Structural measurements have indicated relatively shallow dips within this block of rock, with the dip being mostly towards the ESE but some in the opposite direction.

The geological cross-section from the eastern end of Long Ping Station to viaduct piers from 419 to 417 done at the detailed study stage is shown in Figure 2. As can be observed, the first encounter of marble rock is fairly consistent at around -20mPD. This contrasts with elsewhere in Yuen Long where severe faulting and intrusions of granodiorite and rhyrolite, giving rise to very deep rockhead.

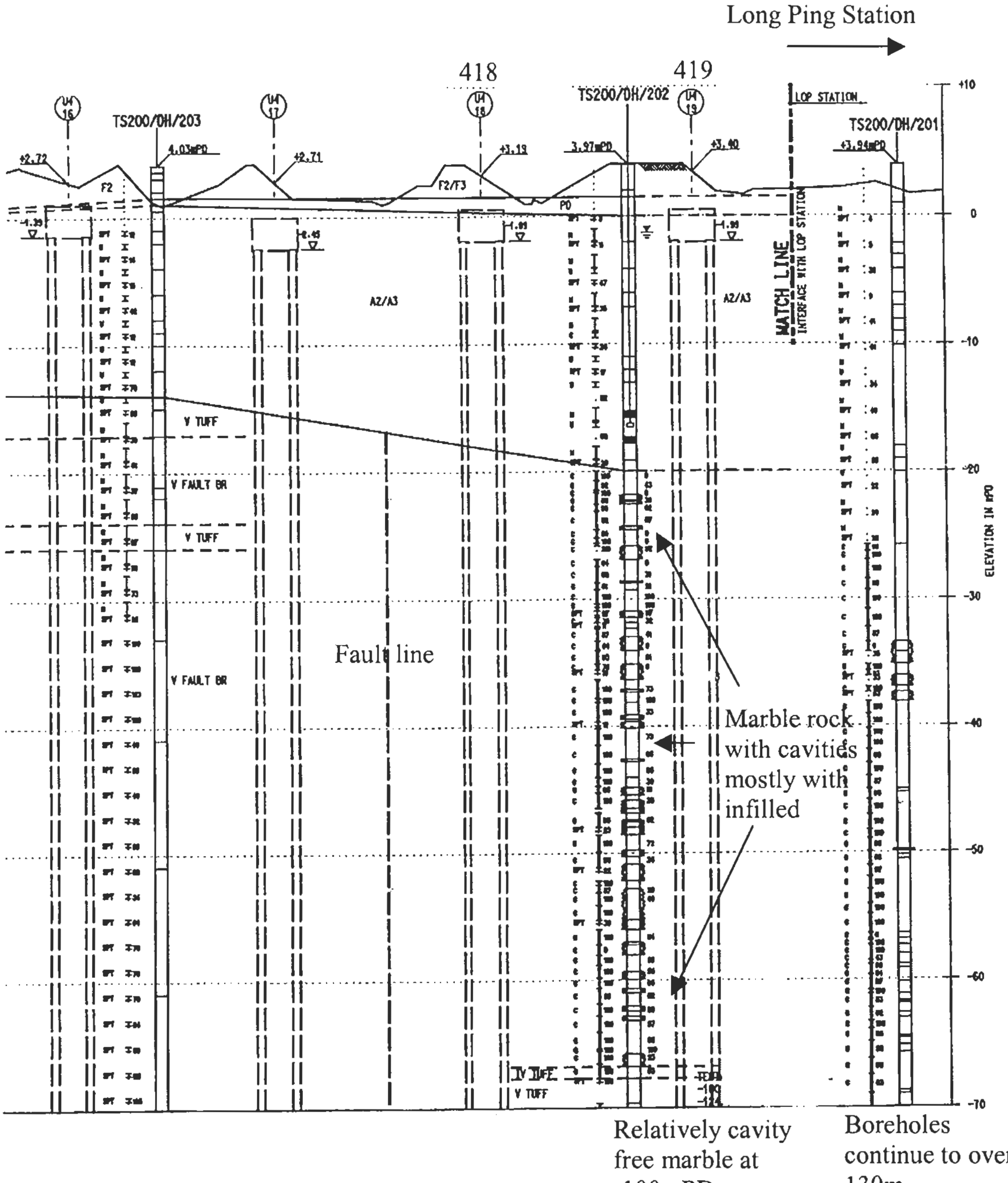

Figure 2 - Geological section showing infilled marble from original SI

CONVENTIONAL DESIGN AND THE ORIGINAL DESIGN

The common types of deep foundation in karstic areas in Hong Kong and their designs have been summarised by Chan (1996). Holmes and Keung (1990) described an instrumented pile loading test with a 0.8m diameter pile end bearing on rock and with strain gauges installed within the rock mass. It was deduced that the stress bulb reached a depth of 8 times the diameter of the pile. For the particular project, 2m diameter piles were selected and together with a contingency of 25%, it was concluded that a depth of 20m of rock underneath each pile should be achieved as one of the design criteria. Daley (1990) reported the use of driven H-pile group with a redundancy of 25% at 3 sites in Yuen Long.

Due to the very tight programme and site constraints from the existing nullah, traffic and major utilities all in close proximity to the proposed foundation, each column of the portal was designed to be supported by a single large diameter bored pile with loads of up to 28MN. Based on existing practice and without the support of full-scale testing, it was required to found the piles at a level relatively free from cavities. This implies that piles of up to 2.5m diameter would have to be installed down to around 100m below ground level. During installation, the excavation would have to be carried out through interbedding of rock and cavity infilled layers. A permanent steel liner would then be placed over the section with cavities prior to concreting to ensure a high quality pile shaft is achieved. The construction is therefore complicated and difficult, requiring several weeks to complete each pile. From the consideration of the programme, the extreme construction difficulties and hence risks, the contractor proposed an alternative.

THE PROPOSED ALTERNATIVE

As the first encounter of rockhead is around -20mPD, it is not possible to utilise the upper layer of soil friction using floating bored piles to carry the large vertical loadings. The only other alternative would then be the utilisation of the friction from the rock using rock socketed piles. There were two major uncertainties in pursuing this design concept. First, the marble rock appears to be densely distributed with infilled cavities from the boreholes. The obvious question is whether the rock is intact. Second, what would be the friction design parameters? These, together with the construction method, are addressed in the following paragraphs.

Geological Model

It is necessary to have a good understanding of the marble rock and the formation of cavities in order to postulate the appropriate geological model for the proper design of the foundation. The existing geological map (1: 5000), prebores for Long Ping Station and the viaduct foundations and all existing boreholes from the site investigation stage of the West Rail projects were gathered and reviewed closely. The more recent boreholes are in fact showing a more complex geology than that reflected in the geological map. However, it can be inferred from the boreholes that the marble encountered in the pier locations of 418 and 419 are common with that appearing beneath Long Ping Station. Despite the succession of infilled layer shown in the ground investigation logs near piers 418 and 419 (Figure 2), it was considered unlikely that this represents marble lumps floating in a soil matrix such as that commonly found in granitic environment. It is important to visualise the infilled materials within the rock mass, as located by the boreholes, in a 3-dimentionsal manner as a distribution of inclined joints rather than infilled materials in horizontal layers. This view was later reinforced by a report produced by Fletcher (2002) after the completion of the pile installation.

Design Approach

Having established that the infilled marble rock at the concerned pier locations is a massive rock outcrop, it was logical then to proceed with the alternative design using the rock socket. Rock socketed piles are common in Hong Kong in the forms of bored pile, pre-bored H-piles and minipiles. The allowable friction between the concrete and rock socket commonly adopted for design is one-tenth of the allowable bearing stress. For Grade III granite/tuff, the allowable friction would be 500kPa. However, since no data is available for the friction parameters to adopt in the design of cavity infilled marble, a full-scale trial pile was proposed prior to the construction of the working piles.

It was also recognised that confinement to the marble rock would be important in the utilisation of the shaft friction when there are cavities, infilled or otherwise, within the rock mass. To ensure this, post construction shaft grouting was adopted prior to the application of the column, deck and live loads. The following sections provide some construction details for the successful application of this alternative design.

CONSTRUCTION CONSIDERATIONS

In conventional bored pile construction in Hong Kong, a temporary casing is installed to rockhead to prevent the collapse of the bore. This immediately presents a problem when dealing with this situation of the infilled marble rock here. Once rock is encountered at around -20mPD, it would be extremely difficult to advance the casing any further. The use of permanent liner is often adopted when dealing with large and voided cavities in order to ensure the quality of concreting. However, this technique is unable to help in this case in preventing the collapse of infilled materials in cavities. The Contractor, for this particular case, is familiar with the technique of bored piling under bentonite. Such technique, with some modifications, was found to be well suited to the pile construction in these conditions.

Excavation Under Bentonite

In this technique, the excavation of the overburden was by means of rotary drilling, supported by bentonite. The least stable upper portion of the bore was still supported by a temporary casing for a length of up to 12m. In order to reduce the risk of losing excessive bentonite into the cavities in the marble rock, and to provide additional strength to the infilled materials, pre-grouting was carried out at five points (4 points for the 1.5m trial pile) around the perimeter of each of the proposed piles before the start of overburden excavation. At each pile location, four evenly spaced boreholes around the perimeter of the pile were drilled and logged to determine the depth of pre-grouting. The grouting was applied using plastic Tube-a-Manchettes (TAM) from a depth of 10m below the founding level of the pile to the first rockhead. A cement mix was used with grouting pressure of around 40 bars as one of the termination criteria. Grouting was also stopped when the intake at each lift of 300mm exceeded 25m^3 even if the targeted pressure was not reached. The process was then repeated after approximately 4 hours until the pressure criterion was achieved.

Shaft-grouting

The technique for shaft-grouting here is similar to that for shaft-grouted piles or barrettes utilising friction in soils. Tube-a-Manchettes , consisting of 50mm diameter steel pipes of 2mm wall thickness were attached to the reinforcement cage of the pile. The number of TAM pipes per pile will vary and for the 1.5m diameter trial pile, 4 pipes were installed. After casting the pile, the TAM pipes were checked and flushed, if necessary, to ensure clear passage. The manchettes, with longitudinal spacing of 0.5m, were then cracked open approximately 12 hours after the completion of concreting to create paths for grouting outside the perimeter of the pile.

The zone where shaft-grouting should be applied was determined from the pre-grouting drillholes discussed earlier. The whole rock socket from toe to first encounter of rockhead was grouted. The grouting was carried out using double packers, connected to a grouting pump with a maximum operating pressure of 60 bar. A volume-controlled specification was adopted in which 25 litres/m^2 was applied. The grout was specified to have a Marsh Cone viscosity of 30-50 seconds and able to maintain workability for 30-40 minutes.

PROPOSED FULL-SCALE LOAD TEST

In order to verify the design philosophy of this alternative, a full-scale loading test was carried out by the contractor using a 1.5m diameter pile. The locations of the pile test and piers 418 and 419 are shown in Figure 1. The logs for the centre prebore and the 3 perimeter drillholes for pre-grouting are presented in Figure 3(a). As can be seen, the layering of the marble rock and infilled cavities vary significantly across the small distance of the centre and perimeter prebores. This is typical of the karst geology demonstrating the 3-dimensional nature of the cavities. The logs were studied and it was deduced that the infilled marble rock is indeed the same as that under piers 418 and 419.

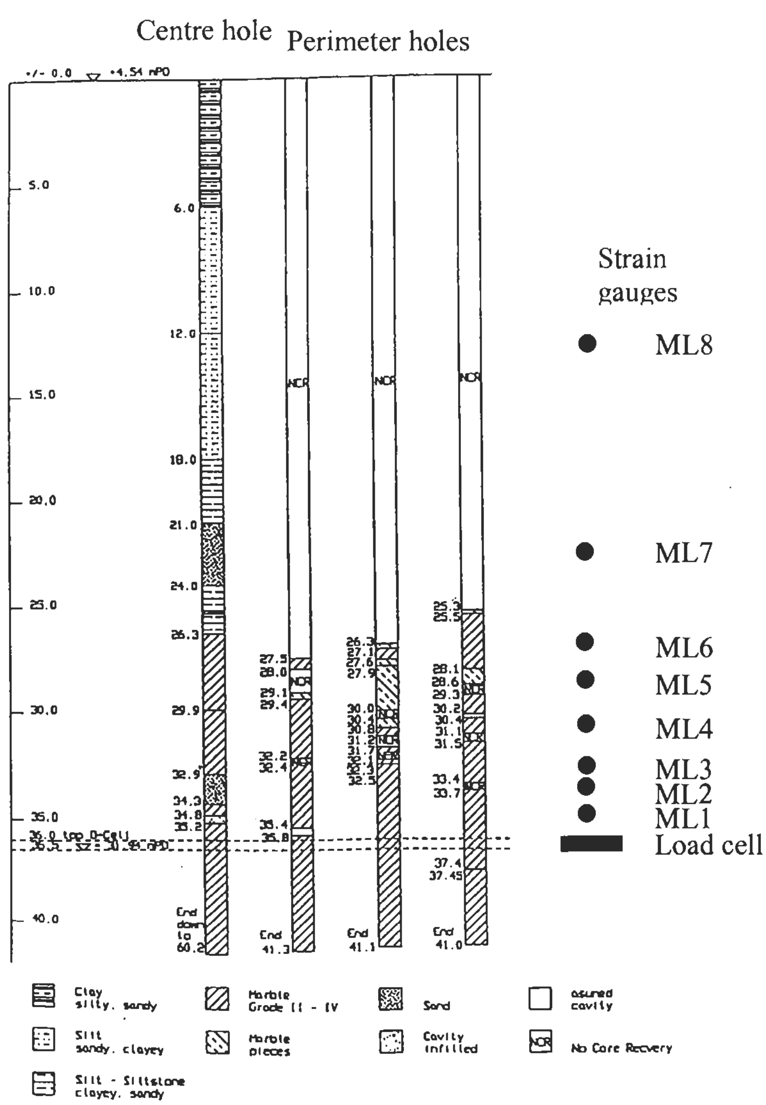

Figure 3 – (a) Prebore at trial pile location (b) Locations of strain gauges

The Test Configuration

The pile was terminated at a depth of 36.5m below ground level and an Osterberg load cell (O-cell), 874mm diameter, was placed between 36m to 36.5m. It was decided to use an O-cell instead of conventional kentledge with shaft sleeving because in this way, the load transfer into the tested portion of the ground (ie the rock socket with cavity infills) can be assured. Also, the interference from the end bearing component of the pile can be completely eliminated, making the interpretation of the test results much simpler and more accurate. The pile ultimate capacity (f_{su}) was estimated based on the following assumption:

$f_{su,rock}$ = 1000 kN/m^2 from marble rock

From this, with an assumed working load of 13,500 kN, a trial pile of 1.5m diameter with a founding depth at 36m was deduced based on the thickness of the marble content encountered in the centre prebore log.

The full test load was therefore set at 27,000 kN. The full test load was achieved in 3 cycles at 50%, 100% and 200%, held for 2 hours, 14 hours and 24 hours respectively, with unloading at each stage. When each load cycle was completed, the pile was left with no load for at least 1 hour before reloading.

In order to provide a stable reference beam, H-piles were installed to support the 12m long beam. To eliminate thermal effect, one end of the reference was allowed to slide relatively freely.

Instrumentation

The pile was fully instrumented with strain gauges, linear potentiometers, extensometers and dial gauges with their locations carefully planned along the depth of the pile.

Two rod extensometers were installed to the top plate of the O-cell and a linear potentiometer was placed across the top and bottom plates to measure the movement of the O-cell. At the pile head near ground level, four linear potentiometers were attached to the stable reference beam to determine any pile head movements. Although the reference beam was supported by piled foundation, its movement was also checked frequently by optical surveys and the movements recorded by the linear potentiometers were corrected accordingly.

In-between these extreme ends of the pile and within the pile shaft, the internal movements were recorded using 20 strain gauges at 8 levels (see Table 1). A higher concentration of gauges was placed towards the socket length in order to provide more reliable data within this critical portion of the pile. The load from the O-cell was measured using a vibrating wire pressure transducer which had been calibrated just prior to the test. The distribution of the straingauges is illustrated in Figure 3(b) relative to the geology from the prebore drillholes.

Table 1 - Distribution of strain gauges along the trial pile

Strain gauge no.	Installed depth (m)	Interpretation zones	Soil/Rock description	Zone no.
ML8A - ML8B	12.64	ML8 - ground	Mainly silty - clayey soils	Z5
		ML7 - ML8	Mainly silty soils	Z4
ML7A - ML7B	22.64			
		ML6 - ML7	Mainly sandy - silty soils	Z3
ML6A - ML6B	26.84			
		ML2 - ML6	Mostly marble rock with infilled materials	Z2
ML5A - ML5B	28.74			
ML4A - ML4B	30.64			
ML3A - ML3B	32.64			
ML2A - ML2B	33.74			
		Top of O-cell - ML2	Mostly marble rock with infilled materials	Z1
ML1A - ML1D	34.84			

Test Results

The test load was first applied on 27 February 2001 and completed on 1 March 2001. All instruments managed to survive through the concreting process and produced sensible readings. Typical readings from the strain gauges were selected and plotted in Figure 4. The strain values from each level (ML1 to ML8) were first averaged and then converted into forces by assuming a uniform cross section area and stiffness along the length of the pile. By taking a linear variation of forces between the different zones assigned in Table 2, the frictional resistances are determined and plotted in Figure 5. As observed, the maximum frictional strength achieved was 970 kN/m^2 in zone Z1. This was eased off slightly to around 917 kN/m^2 as the pressure from the hydraulic system was relaxed marginally. At this load, the top of the pile was measured to have a movement of 8.28mm.

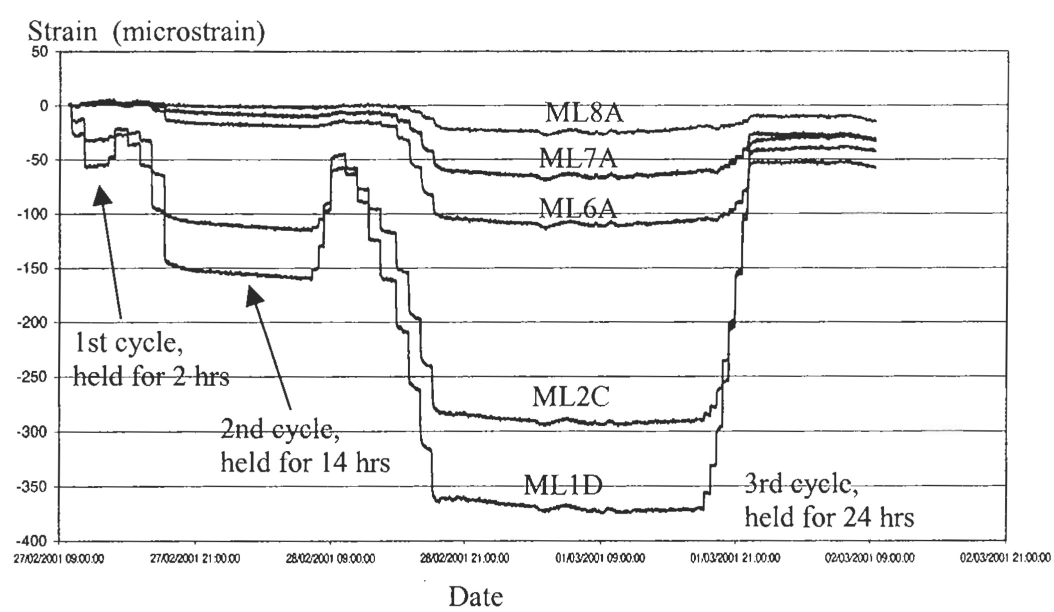

Figure 4 - Strain recorded from selected gauges against time

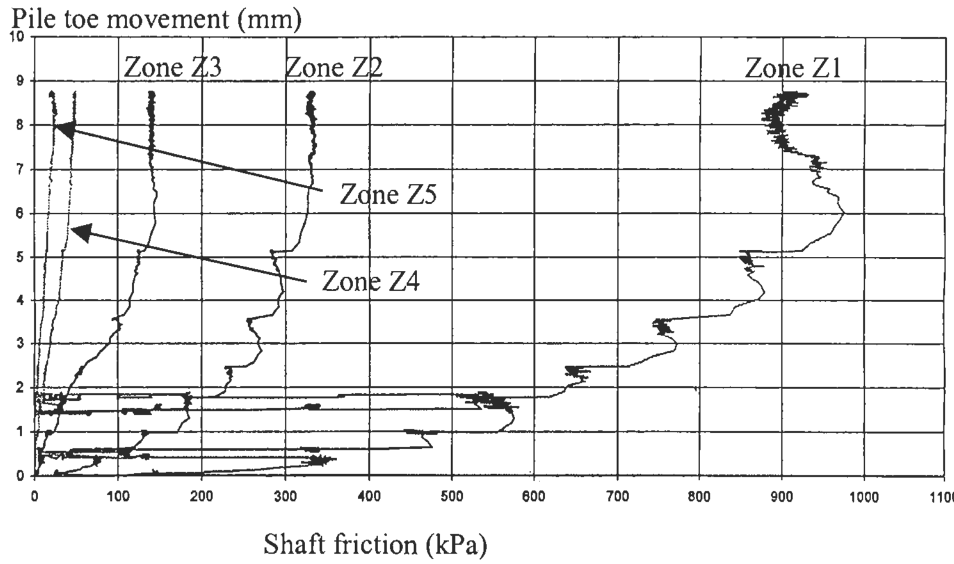

Figure 5 - Mobilisation of shaft friction at different part of the pile shaft

By plotting the frictional strength against the applied load (Figure 6), it can be observed that at the end of the 200% working load, there is no sign of the ultimate strength being reached. The thickness of marble encountered is only 70% of the depth of zone 1 so the actual frictional strength for the marble rock would be around 1350 kN/m^2. It was therefore proposed to adopt an ultimate shaft friction value of 1000 kN/m^2 and with a factor of safety of 2, the allowable design rock socket friction in the marble rock with cavity infills could be taken as 500 kN/m^2.

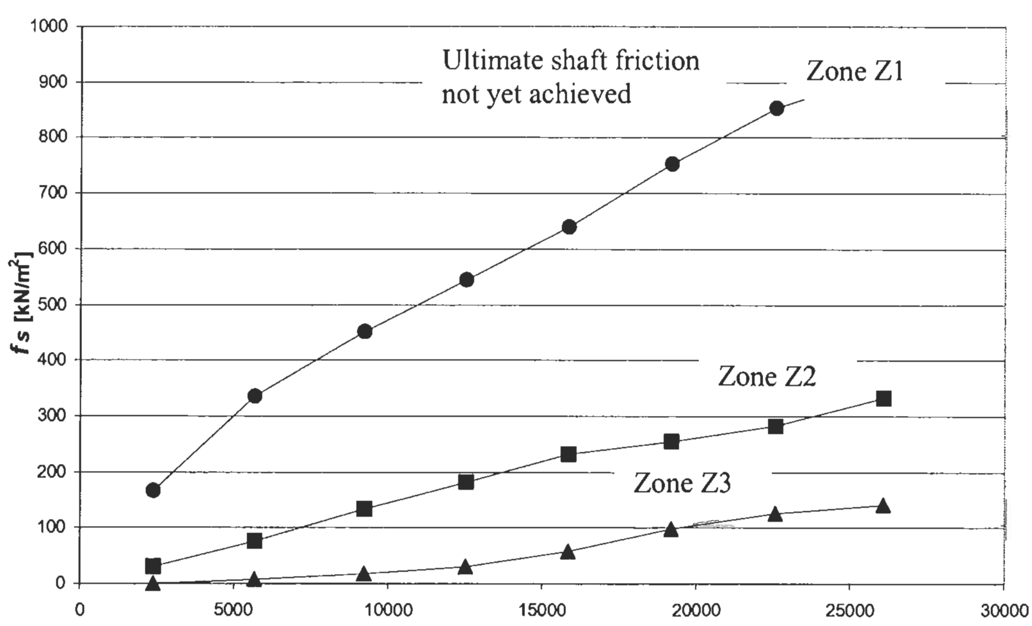

Figure 6 - Ultimate shaft friction

APPLICATION

The trial pile managed to determine the frictional strength of marble rock with infilled materials. The designer then needs to consider an appropriate factor of safety and how the strength should be applied to each pier location case by case. At piers 418 - 419, a comparison of the length and diameter of piles for the original and alternative designs is given in Table 2. The alternative, although required a separate trial pile, managed to reduce the pile length significantly and in fact, reduce the risk at the same time because of the extreme difficulties in constructing very long piles in marble rock with extensive infilled cavities. The trial pile required for the alternative design was well-planned at an early stage and a significant overall saving in construction time resulted.

Table 2 - Comparison of the original and alternative designs

Pier no.	Original design		Alternative design	
	Pile diameter (m)	Pile length (m)	Pile diameter (m)	Pile length (m)
U418	2.5	45.3	2.5	30.8
D418	2.5	61.0	2.5	32.7
U419	2.0	100.6	2.5	31.7
D419	2.0	124.3	2.5	43.8

CONCLUSIONS

In this paper, a case history of a successful application of a full-scale piling test to provide an economical pile design has been described. The application of this design approach must be carried out with caution with careful consideration of the geological model, piling technique, workmanship, load transfer mechanism and allowable differential settlement of the particular structure. It is important for the designer to collaborate with an experienced geologist with good knowledge of the geological setting in the area from the beginning of design, preferably during the ground investigation stage. The designer must be convinced that the test pile and the working piles are located within the same geological formation with very similar characteristics in terms of the depth of first encounter of rockhead, infilled materials and rock types.

REFERENCES

Arup (1998). "Final Geotechnical Data Report." KCRC West Rail Project, Yuen Long Section, Agreement No. DD-200. Ove Arup & Partners Hong Kong Limited, 1998.

Chan, Y.C. (1996). "Foundation in Karst Marble in Hong Kong." Proceeding of the 12th Southeast Asia Geotechnical Conference, Kuala Lumpur, Vol. 2.

Daley, P. (1990). "A Review of Pile Driving Records at Three Sites in Yuen Long." Proceedings of the Conference on Karst Geology in Hong Kong, Hong Kong, pp 123-134.

Fletcher, C. (2002). "Geological Investigation of Bored Piles at Grid 418/419 and Test Pile for Viaducts near Yuen Long Estate." Arrow Geoscience Limited. Report produced for Baurer Hong Kong Limited.

Frost, D.V. (1992). "Geology of Yuen Long." Hong Kong Geological Survey Sheet Report No. 1, Geotechnical Engineering Office, CED.

Holes, D.G. & Keung, C.P.Y. (1990). "Design for foundations in karst limestone." Proceedings of the Conference on Karst Geology in Hong Kong, Hong Kong, pp 107-113.

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將軍澳 55(A)填海區地鐵 601 明挖隧道及房屋基礎的施工

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Construction of MTR 601 Cut and Cover Tunnels and Development Foundations in Reclamation Fill in Area 55(A) at Tseung Kwan O

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摘要

由於將軍澳 55(A)區具有相當厚度的海泥，在其上建造深基礎、設計和施工都有相當的難度。在此填海區建造一段明挖地鐵隧道，設計上採用壓力灌漿技術加固 3 米厚的海泥，以減少開挖引起的地下連續牆的位移。在開挖隧道底板及頂板澆築及回填的施工過程中，對地下連續牆進行了全面的監測，取得了地面沉降，地下水位及土層和岩層的孔隙水壓力變化的數據。結果表明，將軍澳的深層降水會引起填海區明顯的沉降，而對地下連續牆及灌注樁的影響甚微。明挖施工降水受場地外補給的影響，反引起局部水位的少許變化。隧道內的滲流用止水材料控制，而其他小的滲流區域則依情況處理。基底開挖時並未產生明顯的上拱變形和滲流。相隔隧道接近時，因受相隔地下連續牆及樁基的影響，難以採用地錨支撐體系，而採用內撐體系支撐地下連續牆。待明挖隧道結構施工完畢後，才開始施工隧道側及隧道間的樁基，以盡量減低對連續牆穩定性的影響。實踐証明，這些施工方法對填海區高密度房屋基礎之建造是相當成功的。

Construction of MTR 601 Cut and Cover Tunnels and Development Foundations in Reclamation Fill in Area 55(a) at Tseung Kwan O

Nigel R Wightman[1] **& Chris K. W. Cheung**[2]

ABSTRACT: The Tseung Kwan O reclamation has posed difficulties for the design and construction of deep foundations and tunnels due to a significant thickness of marine deposits underlying the site in Area 55 (a), Phase 1 and 2B. The tunnels for the MTRC railway converge within the site to one tunnel section prior to entering the Tseung Kwan O Station farther to the west. The design of the diaphragm walls for the tunnel required the marine deposits to be jet grouted to provide a stiffer layer of soil 3m thick to reduce possible movements of the diaphragm walls during the subsequent excavation works. The diaphragm walls were monitored for performance during the excavation, base slab and roof slab construction and finally backfilling. Monitoring records of settlement markers, standpipes and piezometer records of water levels in both soil strata and rock were obtained. The very deep drawdown of the water levels in the Tseung Kwan O area lead to significant settlements of the reclamation fill but with minimal affect of the performance of both diaphragm walls and bored piles. The construction process of dewatering for tunnels excavation showed that the drawdown was controlled by source exterior to the site but locally levels were affected but to a lesser degree. Seepage into the tunnels was controlled by using water bar materials and other small areas of seepage were dealt with insitu. At the lowest level of the excavation, no base heave was evident and no major seepage observed. With tunnels in close proximity an internally braced prop and wale system was used to support the diaphragm wall as the use of ground anchors was not possible due to the high density of other pile and adjacent diaphragm walls. The bored piling works, close to and in between the tunnels, was constructed later once the tunnels had been completed with minimal effect on the stability of the walls. The construction process for both the tunnels and bored pile foundations in advance, through new areas of reclamation to be used for high density residential use proved to be successful.

INTRODUCTION

The MTR 601 tunnels to the east of Tseung Kwan O MTR Station bifurcated into three separate tunnels (Plate 1). The cut-and-cover tunnels were constructed in advance of foundations for a large residential building under which it passes. The construction works are part of the Phase 1B and 2 for the proposed development at Tseung Kwan O Town Lot 58 in Area 55(a) as shown in the site location plan in Figure 1.

The Tseung Kwan O reclamation area has already been subjected to major settlements with nearby housing developments being affected. The construction control of the tunnels and building foundations was to create no noticeable further settlement to adjacent properties and

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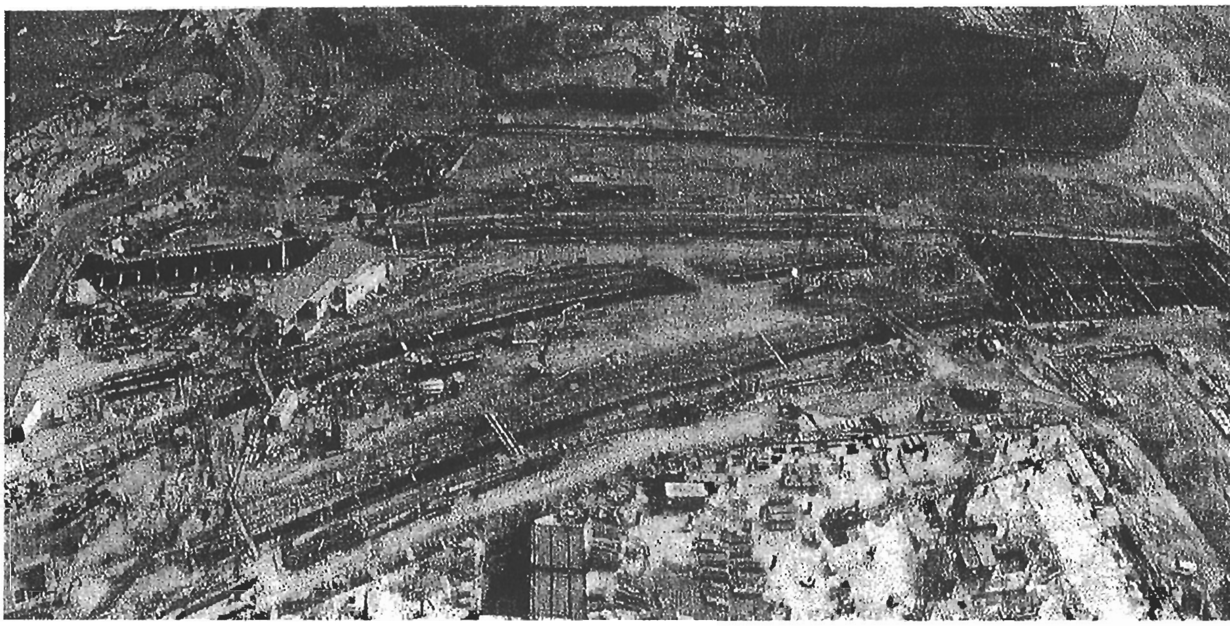

Plate 1. Aerial view of the tunnel works in progress.

ground. The performance of the diaphragm walls during excavation and tunnels base and roof construction was carefully observed by instrumentation. The affect of the subsequent construction of the piled foundations was also minimized due to the proximity of the tunnel walls and other nearby constructions.

DESIGN IMPLICATIONS

The design of the tunnels in reclamation required the underlying marine deposits to be strengthened using a jet grouting method (Figure 2). Diaphragm walls were proposed which extended through the reclamation and founded onto rock. Figure 3 shows a plan of the bifurcating tunnels. The main issue was wall stability and with the presence of weak marine deposits providing low passive resistance. The predicted movement of the diaphragm walls was

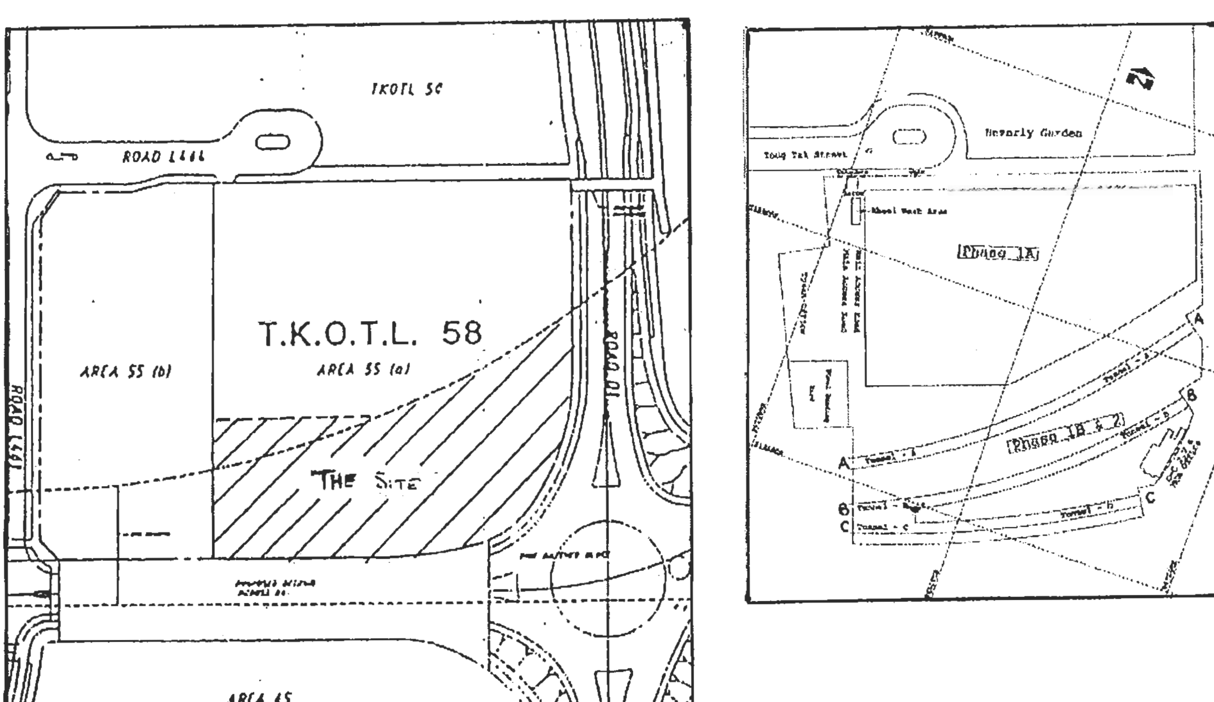

Figure 1. Location of Site 1B and 2 in Area 55 (a) in Tseung Kwan O.

deduced considerably by the introduction of the stiffer jet grouted marine deposit layer. The computer program DIANA was used to analyse the diaphragm wall at various stages of propping and excavation. The design properties for the 3m thick jet grouted layer were undrained shear strength, c_u, of 500kPa with an elastic modulus, E′, of 200MPa.

The ground conditions required for both the diaphragm wall and large diameter bored pile foundation were grade II to III rock with a total core recovery of greater than 85%. The diaphragm walls were 6.2m wide and keyed into the top of the rock by 500mm and the 3m diameter bored piles founded with a 3m deep socket with a bell out to 4.2m diameter. The bored piles were located within 500mm of the diaphragm wall panels as shown in Figure 3.

GEOLOGY AND GROUND CONDITIONS

Solid Geology

The solid geology of the site comprised a volcanic rock of fine ash Tuff with weathering grades from completely composed to slightly decomposed Tuff.

Superficial Geology

The superficial geology comprised reclamation fill overlying marine deposits overlying the alluvial deposits overlying the solid geology.

Ground Conditions

The reclamation fill consisted of granular fill with building rubble. The marine deposits were generally a soft dark grey Clay and the alluvial deposits consisted of a brownish grey Sand overlying a sandy sub-angular Gravel. The ground improvement to the soft marine deposits prior to tunnel construction consisted of the installation of vertical wick drains prior to the reclamation fill being placed. The marine deposits were further improvement by jet grouting which was designed to

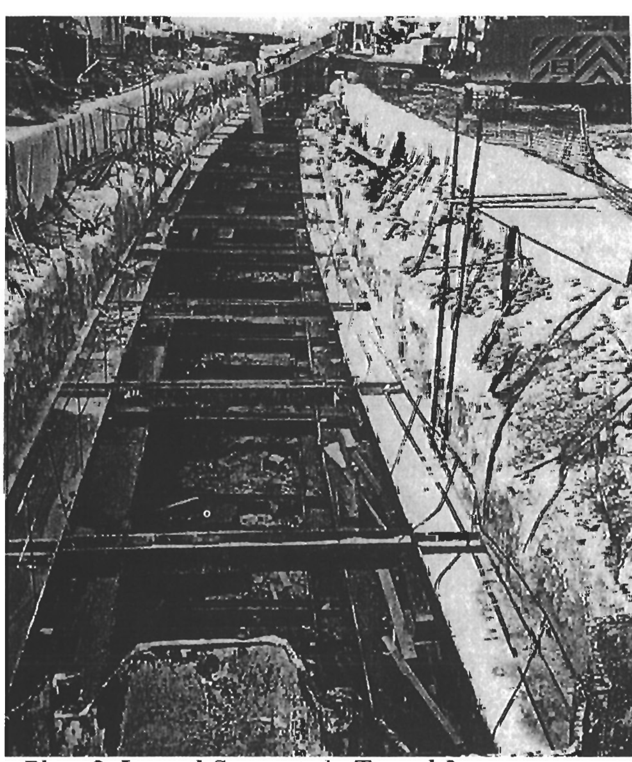

Plate 2. Lateral Supports in Tunnel 3

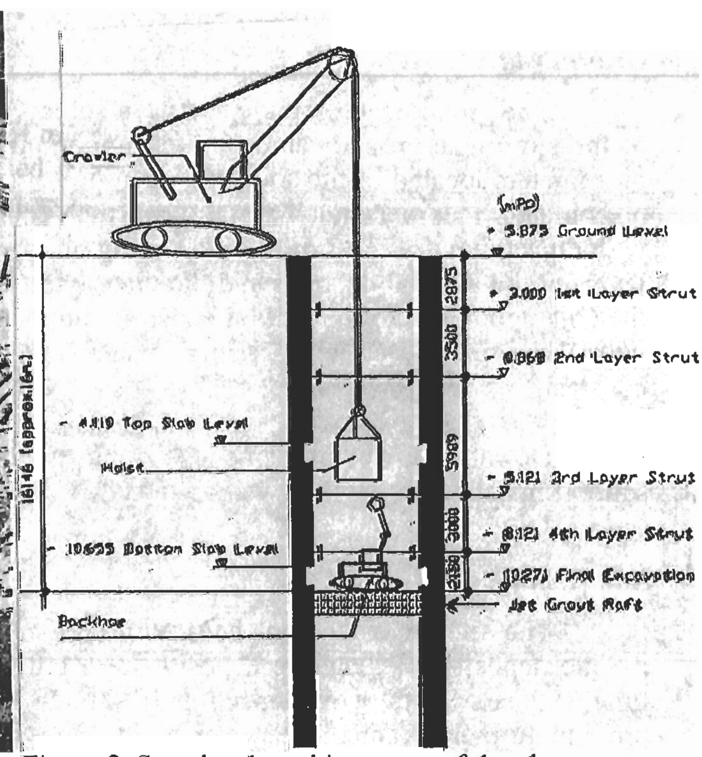

Figure 2. Strut levels and jet grout raft level

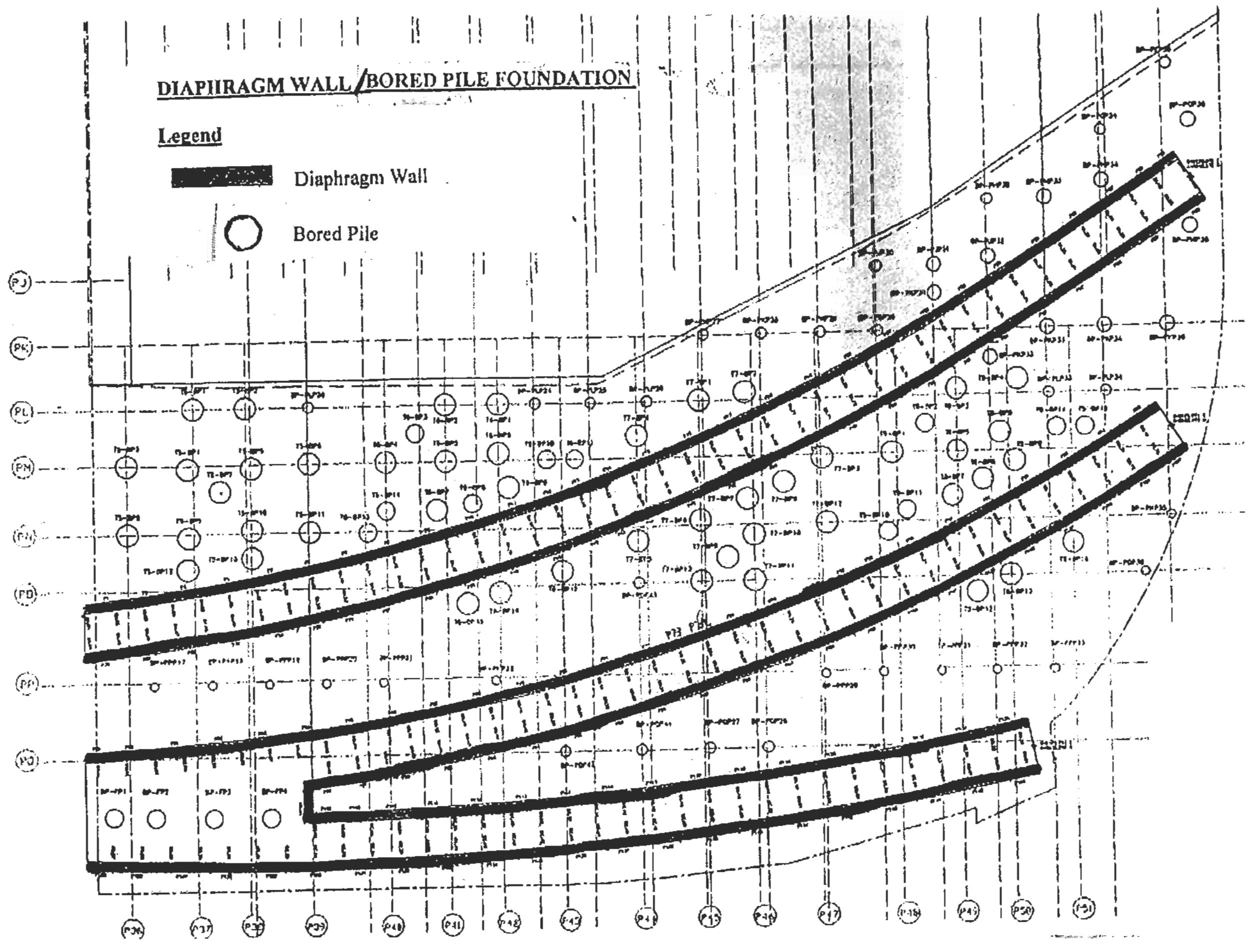

Figure 3. Location of Large Diameter Bored Piles and Diaphragm Walls.

significantly increase the strength of a 3m thick layer to help support the diaphragm walls.

Groundwater

The groundwater regime had been subject of much influence from outside the boundaries of the site. The groundwater monitoring comprised both standpipes and casagrande standpipe piezometers located in the soils and deep piezometers located in the underlying rock mass. The piezometers recorded much variation in the ground water level during the construction period with drawdown in excess of -22mPD being observed within the rock mass. This caused the water levels in the alluvial deposits and the underlying strata to be substantially drawn down. Prior to the completion of the foundation construction phase groundwater levels responded to a recharge with the groundwater levels rising to approximately –5mPD where they remained relatively constant.

Dewatering was implemented within the tunnels sections prior to excavation and lateral support installation.

Site History

The site history commences with reclamation of Junk Bay by end tipping fill over the marine deposits. The marine deposits were left insitu and have consolidated over a period of time. (8 years having commenced in 1992). In 1996 the areas of the roads were surcharged to provide further loading to accelerate the consolidation process and reduce settlements later during road construction. The reclamation area was monitored for settlement prior to construction over a period of 2 years with settlements up to approximately 1,000mm being recorded. During the tunnel construction period a further settlement of approximately 309mm was recorded at the Phase 1B and 2 site. A recently completed study has indicated that excessive settlements had been caused by dewatering of the underlying decomposed volcanic rock layer causing compression to be experienced below the marine and alluvial deposit layer, (Maunsell, 2000).

CONSTRUCTION DETAILS

Construction Planning

The order of construction for the MTR 601 tunnels was:

a) Diaphragm wall and instrumentation,
b) Jet grouting a raft,
c) Excavation and lateral support,
d) Base slab and water proofing,
e) Roof slab and water proofing, and
f) Backfilling to ground level.

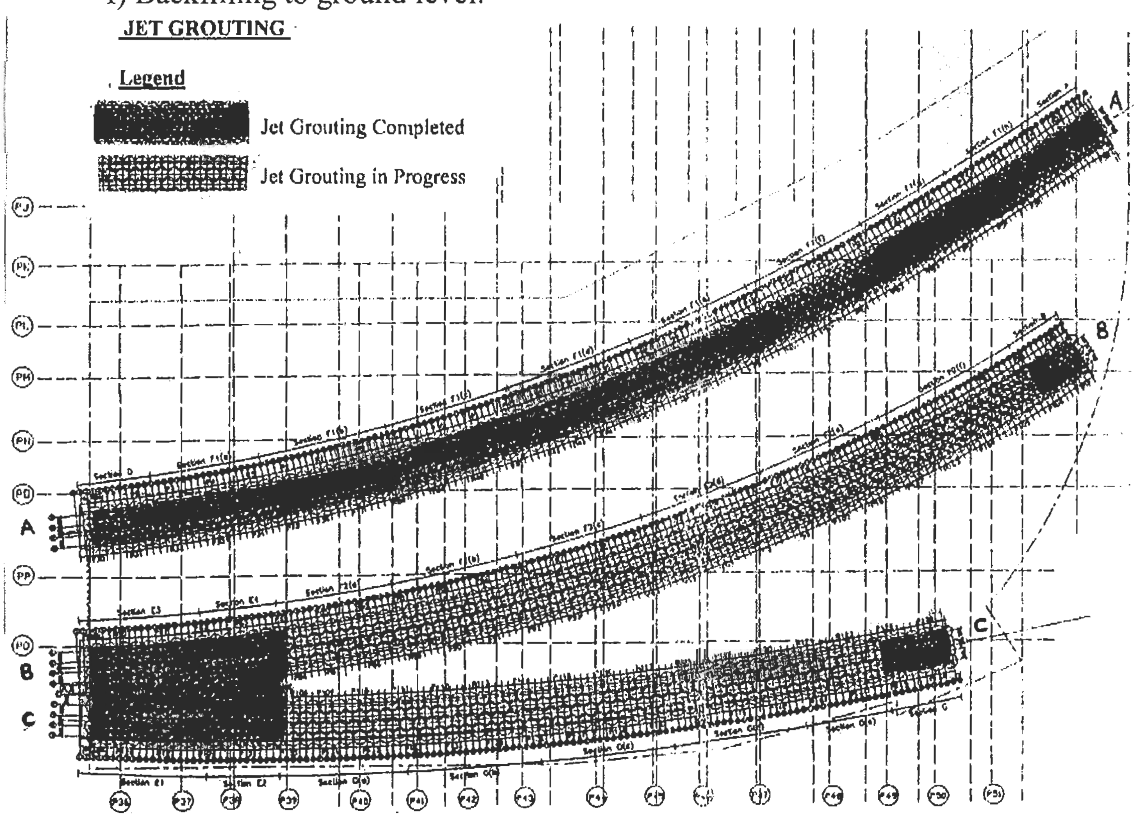

Figure 4. Layout of Jet Grouting Between the Diaphragm Walls in Tunnels 1, 2 and 3.

Construction monitoring

The settlement and groundwater monitoring continued throughout the construction phase for the tunnels and foundations. The ground settlement data indicated that the consolidation process had not completed with ground movements continuing to be recorded after completion of construction at a rate of up 13mm per month being recorded. The affect of the works on ground settlements proved difficult to conclude but with exception to either rapid movement of groundwater level or adjacent temporary works support systems the downward trend was progressive and constant.

The settlement profile from one such ground monitoring station indicated that the consolidation process would continue. The groundwater level profile also indicated highly variable groundwater levels experienced during the construction of the tunnels. It was therefore important that the bentonite slurry mix remained at a constant level within the guide walls during the construction of the diaphragm walls to enhance stability of the excavation.

Jet Grouting

The jet grouted raft was constructed from ground level using specialized high pressure injection equipment and mixer to stiffen the upper three metres of the marine deposit layer. The formation of the base slab of the tunnel coincided with this layer. Figure 4 shows the layout of the jet grouting locations relative to the tunnel alignment. The jet grouting was carried out on a triangular grid so that the adjacent grouted zones were intersected.

Diaphragm Wall Construction

The construction of diaphragm walls involved the construction of guide walls to contain bentonite slurry. The diaphragm walls were reinforced with the insertion of reinforcement cages prior to concreting. The diaphragm wall panels extended to rockhead (grade III or better decomposed volcanic rock). Water bars were inserted between each panel to ensure leakage into the tunnels was minimized. Inclinometer tubes were installed in 4 panels to allow for monitoring of the wall panel during subsequent dewatering, excavation and lateral support installation. The individual panels for the three tunnels joined at the merging points, which were constructed with diaphragm walls to form a large box section.

Excavation and Lateral Supports

Excavation of the tunnels commenced to 0.5m below the first level of lateral supports (Figure 2). The frame work was erected using standard section props and wales. The second, third and fourth stage of excavation similarly was carried out to 0.5m below the level of lateral support required to be installed. The excavation was completed at approximately the top level of the jet grouted marine deposits. There were no soft spots observed within the jet grouted layer confirming that the ground improvement method was successful.

Tunnel base and roof construction

The jet grouted layer was stiff and provided an adequate surface to lay the waterproofing membrane and steel cage for the lower tunnel slab. The construction of the base and roof slabs was carried out in sections with water bar placed between sections. The base and roof slabs were keyed into the diaphragm wall to provide a sound connection. The top of the roof slab was sealed against water ingress prior to backfilling with fine granular fill compacted in layers. The lateral supports were removed during the backfilling process. The inclinometer instrumentation was monitored throughout the tunnel construction to ensure the panels were not disturbed and that the ground treatment was providing adequate lateral resistance.

Adjacent foundation construction

The foundations of the residential development located over the site commenced once the tunnels had been completed. The 2.5m diameter piles were bored under bentonite slurry head in the Phase 1B and 2 site.

DISCUSSION AND CONCLUSIONS

Construction monitoring

The ground settlement levels, groundwater levels and the inclinometers were monitored on a regular basis, daily and weekly respectively. The review of the groundwater levels showed that severe depressions in the groundwater levels were not affecting the diaphragm wall construction nor the performance of the completed walls. The large diameter piles appeared to affect groundwater levels over a small radius and no affect on the diaphragm walls was noted.

The ground levels were regularly reviewed to provide an early warning of sudden changes to settlement trends. Although ground levels were constantly showing continuous settlement it was still possible to monitor the affects of construction on the readings obtained and to react accordingly. The background level of settlement was approximately 20mm to 30mm per month during the construction period. Occurrences of deviation occurred from the adjacent construction site of Phase 1A during the lateral support installation of the deep basement temporary works. The rectification of the causes of settlement were implemented quickly by props being installed to prevent the perimeter wall deflection further.

Diaphragm Wall Performance

The diaphragm walls were unaffected by the low groundwater regime. The adjacent construction sites had little affect during the tunnel construction nor post construction performance. Recharge of the groundwater came at a later time after tunnel construction but before large diameter bored pile construction commenced. Some limited seepage into the tunnels was noted through some joints between diaphragm walls and these were sealed using propriety products of expanding foams to prevent further groundwater ingress.

The inclinometer readings for panel number P140 on the outer wall of Tunnel 3 (Figure 5) show approximately 12mm of lateral movement at a depth of 12m. The lateral movement during the excavation phase was within acceptable limits for the construction of the tunnels. The design of the diaphragm wall predicted movements of 36.9mm at a depth of 28m (Figure 6). The performance of the diaphragm walls was therefore acceptable with regard to the ground improvements made to the initially soft marine deposits that had been left insitu during the reclamation process. The lower bound value of undrained shear strength of 30kPa in the

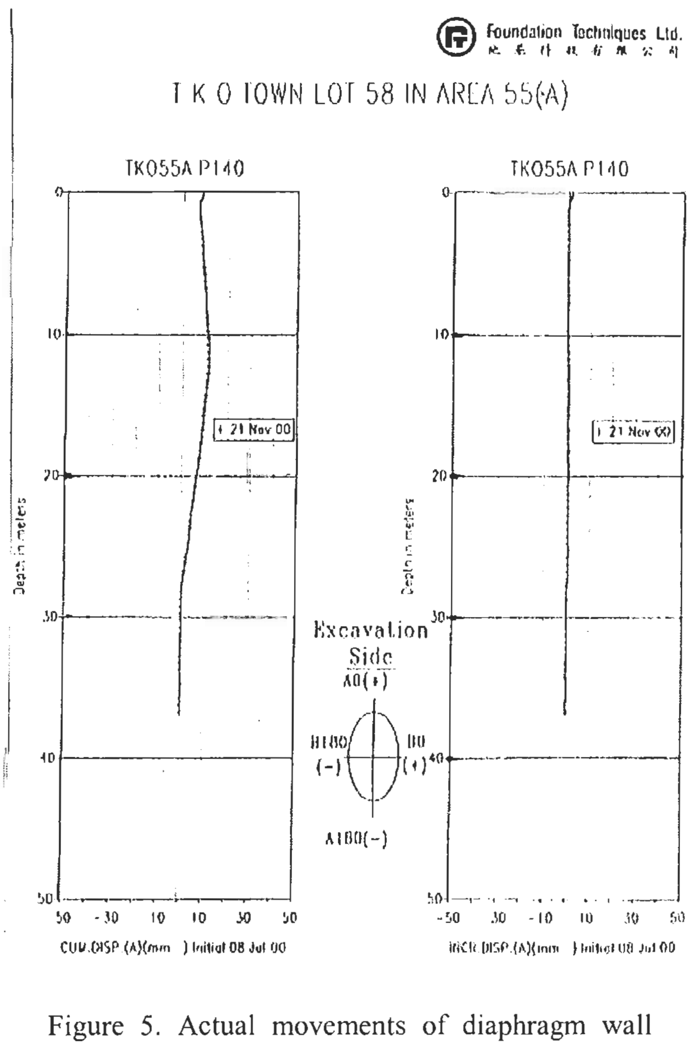

Figure 5. Actual movements of diaphragm wall measured by inclinometer in panel number P140.

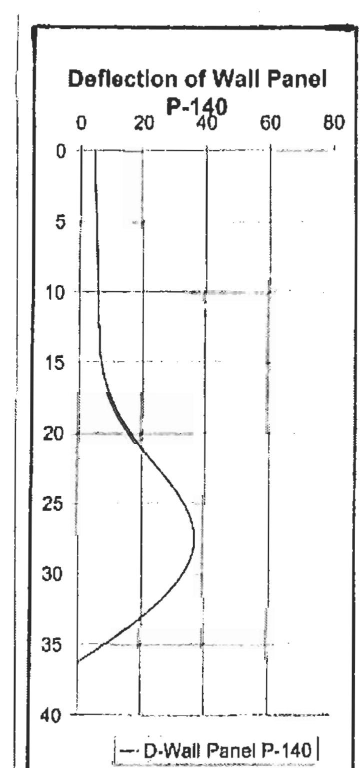

Figure 6. Design movements of diaphragm wall

model can be compared to the average undrained shear strength of 63kPa obtained from in-situ shear vane tests and explains the reduced movement of the diaphragm wall experienced.

Foundation Construction Affects on Diaphragm Walls

The foundation construction do not have any affect on the completed tunnels even though some of the large diameter piles were located within 1m of the diaphragm walls. Locally the groundwater was depressed near the bored pile drilling locations but this recharged once drilling was completed. The construction process for both the tunnels and bored pile foundations in advance, through new areas of reclamation to be used for high density residential use, proved to be successful.

Completion of tunnels

The early completion of the tunnels enabled the MTR 601 tunnels to be worked on simultaneously with the foundation pile cap and superstructure construction for the residential development allowing early completion of the track laying and control systems.

REFERENCES

Maunsell, 2000. 'Investigation of the Unusual Settlement in Tseung Kwan O Town Centre, Territory Development Department, New Territories East Development Office.

ACKNOWLEDGEMENTS

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西鐵 CC300 荃灣段隔膜牆的表現評估

布賴恩 洛克裏夫　　**BACHY SOLETANCHE GROUP LTD.**

ASSESSING THE PERFORMANCE OF DIAPHRAGM WALLS ON KCRC CONTRACT CC300 TSUEN WAN STATION AND APPROACH TUNNELS

Brian A.W. Norcliffe
Bachy Soletanche Group Ltd.

撮要

九廣鐵路西鐵荃灣段是採用明挖方法在地下建造的，明挖施工包括了利用約共 2 公里長的隔膜牆作爲在剛完成填海地區的擋土結構。本工程坐落於硬土上的箱形隧道是採用臨時性隔膜牆，而車站本身則利用隔膜牆作爲永久性結構的一部份。

本篇利用採自自動傾斜儀及傳統傾斜儀得來的數據，用以比較隔膜牆移動的數理預測及實則觀測，並用本篇以下描述的方法檢測隔膜牆結構是否超出使用及極限狀態。

香港工程師學會岩土分部每年都舉行一次學術會議，議題的選擇是環繞香港土力工程界所關心的問題，取錄的論文將會刊登於會議紀錄上，通過篩選的論文作者，會被邀請在學術會議上發表。本文旨在闡明論文的格式要求，並以本文作爲例子，以供作者參考。

ASSESSING THE PERFORMANCE OF DIAPHRAGM WALLS ON KCRC CONTRACT CC300 TSUEN WAN STATION AND APPROACH TUNNELS

Brian A.W. Norcliffe[1]

ABSTRACT: The KCRC West Rail Contract CC300 "Tsuen Wan Station and Approach Tunnels" was constructed by cut and cover techniques, and involved 2km of diaphragm walls built in recently reclaimed land. Temporary diaphragm walls were constructed to allow floating box tunnels to be constructed in the approach tunnels. The station diaphragm walls were incorporated into the permanent station structure.

Vibrating wire in-place inclinometers (IVI's) and traditional inclinometers (INC's) were used to compare actual diaphragm wall movements with numerical model predictions, and to assess whether the serviceability or ultimate limit states were met. The technique used for this assessment is described.

INTRODUCTION

The Kowloon and Canton Railway Corporation (KCRC) has undertaken construction of a railway linking the Western area of the New Territories to Kowloon known as "West Rail". One contract for the railway's construction is CC300: The Tsuen Wan West Station and Approach Tunnels. The railway station (400m in length) and approach tunnels (each 300m in length) were constructed as cut and cover. The main contractor was a joint venture of Penta-Ocean Construction Ltd and Kier Hong Kong Ltd. The retaining system consisted of diaphragm walls with multi-layer propping by compound struts. The diaphragm walls were constructed by a joint venture of Bachy Soletanche Group Ltd and IP Foundations Ltd. Sol Data Asia Ltd was the instrumentation subcontractor.

During excavation diaphragm wall deflections were compared to design predictions. In certain locations design predictions for deflection were exceeded. Instrumentation was used to determine whether a defined serviceability limit state was exceeded or if the structural capacity of the diaphragm wall was being approached.

DESIGN OF THE RETAINING SYSTEM

The design of the 800mm thick temporary diaphragm walls for the north and south approach tunnels was governed by ultimate limit states. There were no long-term corrosion or deflection concerns for the diaphragm walls as an independent floating box tunnel was designed and constructed between the temporary diaphragm walls.

The design of the 1200mm thick permanent diaphragm walls for the station was governed by serviceability limit states. These diaphragm walls were designed to have a

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limited crack width (within the reinforced concrete) as they formed an integral part of the station structure.

The diaphragm walls were numerically modelled to determine the magnitude and location of bending moments, shear forces and deflection during the various stages of dewatering, excavation and construction. Structural design of the diaphragm wall sections was then carried out according to BS8110 (1985).

Instrumentation and Monitoring

Reservation tubes were attached to the steel cages for the diaphragm wall panels, to allow for the installation of inclinometers at approximately 40m spacing (in panels opposite each other across the excavation). Once the panels were cast the inclinometer tubes were installed inside the reservation tubes and grouted in place. The grout was then allowed to cool for a period of four to seven days. The average inclinometer was 40m long. In order to have useful inclinometer data it was essential that no activities that would impose a load on the diaphragm walls (such as excavation or dewatering) be performed until the inclinometer installation was complete and baselines were taken.

Standard and Vibrating Wire Inclinometers

Two types of inclinometer were used to monitor diaphragm wall deflections. The majority of the inclinometers on the seaward side consisted of a standard slotted access tube with readings taken manually every 0.5m using a standard torpedo type inclinometer gauge (INC). Each INC took approximately 45min to manually monitor (depending on the depth of the access tube).

The majority of the inclinometers on the landward side were in-place vertical inclinometers (IVI's), which consisted of a series of linked vibrating wire tiltmeters specified at 2m spacing installed within the inclinometer tube. A manual torpedo reading was taken on each IVI inclinometer before the vibrating wire tiltmeters were installed. Installation of the vibrating wire equipment usually took between four to eight hours. During monitoring each IVI data-logger would take approximately 5 min to download.

Monitoring the Inclinometers

Once the inclinometers were installed and baselines were established, pumping tests were performed to assess the water tightness of the diaphragm walls. A programme of excavating, strutting and dewatering followed the pumping tests. All of these activities induced deflections of the diaphragm walls that were measured by the inclinometers.

The manual inclinometers were monitored daily during critical phases of dewatering and excavation, and typically at intervals of every 3 days at other times. The results obtained from the manual inclinometers (INC's) were repeatable to within 2 to 3 mm over the full depth of the inclinometer. The IVI's were monitored daily by data-loggers. The results obtained from the IVI's were generally repeatable to within 1 to 2 mm over the full depth of the inclinometer.

Using diaphragm wall technique through soil fill frequently resulted in some concrete overbreak. Mechanical breakers were used to remove overbreak in areas where waling beams for the struts were placed, and where the diaphragm walls formed part of the permanent station structure. On occasion, vibrations induced from the removal of the overbreak caused serious disturbances in the IVI readings, such that the sensors required resetting. When the sensors were affected by vibration, the manufacturer stated that recalibration was recommended to maintain high accuracy. On occasion when the sensors risked damage the IVI's were removed. The tube could be monitored with the standard torpedo INC gauge. The standard INC installations proved to be unaffected by overbreak removal activities.

ASSESSMENT OF DIAPHRAGM WALLS

As part of the instrumentation programme, automatic response values were established within the instrumentation database to flag a warning when the diaphragm walls had reached 50%, 70% and 90% of the ultimate predicted deflections. These warnings were called the Alert, Action, and Alarm response values. When these response values were met, deflections were compared to the numerical predictions for the respective stage of construction. On occasion the predicted deflections at a certain stage were exceeded, and further analysis of the inclinometer data was necessary to assess the status of the diaphragm walls and determine whether remedial measures were required.

The Neutral Axis

The distance from the neutral axis to the tension steel was required in order to calculate the stress in the tension steel, and crack width within a diaphragm wall panel. Partial safety factors for materials were left out for the stress assessment, otherwise the crack width and stress levels would have been underestimated. If two diaphragm wall panels of different stiffness deflect by the same amount, then the stiffer diaphragm wall panel is resisting greater loads, and withstanding greater internal stresses.

The As-Built drawings were used to determine the depth to the neutral axis according to BS8110: Part 2, and the flexural stiffness was then calculated.

The Radius of Curvature

Once the flexural stiffness had been calculated, the inclinometer plots were used to calculate radius of curvature for each panel. The IVI plots provided deflection in 2m depth increments, and the INC plots gave deflection values for every 0.5m increase in depth. The radius of curvature was calculated using two methods.

1. The position of maximum curvature on an inclinometer plot was visually located. A cord was drawn from a point slightly above to slightly below this point of maximum curvature on the inclinometer curve. Excess deflection beyond the cord was measured. By assuming that the inclinometer curve was an arc of a circle over a short length, the radius of curvature could then be calculated from geometry.

2. The inclinometer data was transposed and plotted. An equation was fitted to the curve of the inclinometer data using regression analysis. Radius of curvature was then calculated using beam theory.

The stress in the tension steel, crack widths at a specified location and actual bending moments could then be assessed according to according to BS8110: Part 2.

Performance of the Diaphragm Walls on CC300

In general the measured deflection of the diaphragm walls on CC300 corresponded well with design predictions. This was because the majority of the load acting on the walls was due to water pressure, which was relatively well defined. A detailed assessment was necessary in one area of the temporary diaphragm walls in the south tunnels, and one area of the permanent diaphragm walls in the station.

Deflections of the temporary diaphragm walls in an area of the south tunnel exceeded predicted deflections due to a change in construction activity sequence. Using the inclinometers to analyse the diaphragm wall enabled appropriate remedial measures (such as an additional strut, and another change in construction sequence) to be implemented at an appropriate time. The assessment of the diaphragm walls using inclinometers also allowed works to continue in a safe manner when design deflections were exceeded but stresses were well within the capacity of the diaphragm wall.

The permanent diaphragm walls in an area of the station took on an unusual shape, and original design predictions for deflection were exceeded due to an external excavation. The assessment of the diaphragm walls using inclinometers was necessary to ensure that the serviceability limit state was not exceeded, and the results of the analysis assisted with the decision as to whether remedial measures for the concrete surface were necessary.

Discussion

The second method of calculating curvature (curve fitting and then using beam theory) was more time consuming than the first method, but was especially useful if the maximum curvature was difficult to locate on an unusually shaped curve (due to unexpected behaviour of the diaphragm wall). When assessing curvature from inclinometer plots due consideration must be given to the scale differences; deflection (x-axis) is usually in millimetres and depth (y-axis) is usually in meters.

The specified spacing of the IVI's tiltmeters (2m) within the tubes meant that both methods used to calculate curvature gave a less accurate assessment of the diaphragm walls than the standard torpedo INC gave (with readings taken every 0.5m). The curves developed from the standard torpedo INC's were smoother. Closer data points enable a more accurate assessment.

Some other practical issues involved in assessing diaphragm wall performance using inclinometers are that:

- It is essential that there be no deflection of the diaphragm wall prior to taking baselines on an inclinometer. Late inclinometer installations are nearly worthless and can only be used in an observational sense.
- It is convenient if inclinometer data is compatible with plotting software used for curve fitting.
- The distribution of steel within each panel is often not constant throughout its length, so the location of the neutral axis may change with depth.
- If the best estimate of expected behaviour is required, then the expected or most likely material parameters should be used.

CONCLUSIONS

In cases where the numerically predicted deflection is exceeded, inclinometers are essential in determining the curvature of the diaphragm wall, and hence whether serviceability limit states or the structural capacity are being approached. A decision can then be made as to whether it is safe to continue work, or whether remedial measures are necessary.

ACKNOWLEDGEMENTS

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運作公路下的車輛地下通道的岩土設計

竇華仁與溫立斯

百泰工程顧問有限公司

GEOTECHNICAL DESIGN OF A VEHICULAR UNDERPASS UNDER A LIVE HIGHWAY

Huaren Dou and L. H. Swann
Babtie Asia Limited

撮要

作爲掃管笏路擴寬工程的一部份，擬在屯門公路下建一新的車輛地下通道。 此地道須採用地下施工方法從而保證屯門公路交通不受影響。 地道的臨時支護是採用一系列的水平鋼管樁所形成的拱來實現的。 鋼管樁採用撞擊方法打入土中。 再用建在岩基或素混凝土上的工字型鋼拱肋來支撐鋼管形成的拱。

本工程的主要難點是控制路面變形在很小的範圍內及其在地下 1.5M 潛層灌漿的有效性。 本文將討論此地道的設計與施工要求及土與結構互相作用的數值分析結果.

Geotechnical Design of a Vehicular Underpass under a Live Highway

Huaren Dou[1] and Leslie H. Swann[1]

Abstract: A new vehicular underpass crossing underneath Tuen Mun Road is proposed as part of the So Kwun Wat Road Widening Project in Tuen Mun. The construction of the underpass is to be carried out using the tunneling technique with the traffic on the highway uninterrupted. In order to build the permanent concrete lining for the tunnel, a temporary support for the tunnel excavation is required, and this is provided by an arch formed with horizontal pipes. The pipes are to be installed using the pipe ramming method. The pipes arch is then supported by steel ribs founded on rock or mass concrete. The main challenges of the project for the Contractor and the Design Engineer are controlling of the ground movement within the specified limit during construction, constraints of the limited clearance and the potential requirement for grouting within the 1.5m overburden to the highway pavement surface. This paper presents mainly the geotechnical design aspects of the underpass. The paper will discuss analytical results of numerical modeling of the soil-structure interaction using the computer code FLAC, along with design and construction requirements for the underpass.

Introduction

To meet the traffic growth requirement in a housing development area (Area 56) at Tuen Mun District, a road widening work was planned for So Kwun Wat Road to serve the area. The two lane road passes an existing, one of the busiest, highway - Tuen Mun Road via a present two lane underpass. The upgrading of the So Kwun Wat Road to a four lane standard requires a new two-lane tunnel to be constructed only 1.5 m below the live Tuen Mun Road. The Contract stipulates that traffic on the Highway must be maintained uninterrupted at all times during the construction of the new underpass. The construction also requires that a maximum ground movement on the Highway must be less than 20mm.

The proposed vehicular underpass crossing underneath the existing Tuen Mun Road is located approximately 25m to the west of the existing underpass at So Kwun Wat Road in Tuen Mun Area 56 (Fig. 1). The new underpass, approximately 40 m long, is to have 2-lanes to meet the widening requirements of the existing So Kwun Wat Road which will be extended to four lanes (two in the existing underpass and two in the new underpass). The Tuen Mun Road level in this area varies from about 12.6 to 13.5 m relative to Hong Kong Principal Datum (mPD), with a carriageway width of approximately 30 m.

The construction inevitably requires the underpass to be built using tunnelling techniques. The temporary support for the tunnel excavation is provided by an arch formed with horizontal pipes. The pipes were installed using the pipe ramming method. The main challenge of the project to the Contractor and the Design Engineer is to control the ground

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movement within the specified limit of 20 mm during construction. The limited clearance above the tunnel also presents potential practical difficulty in achieving effective grouting to improve the soil within the 1.5 m overburden to the Highway pavement surface.

GROUND CONDITIONS

Ground investigation, along with laboratory tests, was carried out which consisted of vertical and inclined boreholes. The results of the ground investigations indicate that subsurface conditions at the site consist of fill underlain by alluvial deposits. Below the alluvium are decomposed rock and bedrock (granite). An inferred geological section along the proposed alignment of the underpass is shown in Figure 2.

The fill, forming the embankment of the existing Tuen Mun Road, comprises mainly fine to coarse grained medium dense sand with silts, gravel, and cobbles, varying in thickness from 7 to 9 m in areas below the roadway. Alluvium, 1.0 to 1.8 m thick, was encountered below the fill, which consists of medium dense sand with gravel and silt. Completely to highly decomposed granite (classified as Grade V/IV) was encountered below the alluvial deposit and fill. The decomposed granite comprises clayey to silty coarse sand with a thickness up to 2 m in the subject area

Bedrock (granite, Grade III or better) was encountered beneath the decomposed granite at levels interpreted to range from about 2.7 to 3.0 mPD. The bedrock is in a moderately to slightly decomposed state. It is generally moderately strong to strong (UCS=90 to 190 MPa) with closely to medium spaced joints.

Groundwater was measured approximately at a level of 2.5 to 4.0 mPD in the piezometers installed at the site.

CONSTRUCTION REQUIREMENTS

The design brief requires the Contractor shall build the vehicular underpass using the tunneling construction method beneath the live Tuen Mun Road. The method adopted shall permit the construction of the underpass to be carried out without disruption to traffic in Tuen Mun Road above. The settlement or heave of the carriageways of Tuen Mun Road shall be limited to a maximum of 20 mm and to a maximum gradient of 1%. Ground treatment using pressure grouting may therefore be required to improve the soil properties outside the tunnel lining prior to the tunnel excavation, to mainly minimise ground settlement. The Design Engineer must be capable to produce a design that is able to demonstrate that the underpass can be constructed as required.

DESIGN CONSIDERATIONS

Permanent Lining

The original (conforming) design of the underpass was a rectangular box structure. After detailed study of the site conditions and constraints, a permanent reinforced concrete arch structure was proposed for the underpass (Fig. 3), because this would be structurally more "rigid" than the original rectangular shape assuming the same thickness. This arch lining was designed to have a thickness of 500 mm, based on results of the soil-structural interaction analysis. The arch is approximately half way terminated at the road level of about 4.2 mPD which is structurally connected to the base (road) slab of 700 mm thickness that is supported on the bedrock or mass concrete over the bedrock. The lining is 600 mm thinner than the roof slab of the original box structure.

Construction Sequence

A temporary steel pipe arch support for the tunnel excavation is proposed. The arch will be formed by contiguous steel pipes installed horizontally around the perimeter of the tunnel using the pipe ramming method. The pipes installed have an interlocking with each adjacent pipe. This will prevent water ingress and loss of materials between the pipes. After the pipe arch is formed, the required treatment works to stabilise the face soil inside the tunnel perimeter will be performed and the first steel rib support installed at the portal. The excavation for the tunnel is then followed (Fig. 4). The excavation will be carried out using a top-down method. The main temporary steel ribs shall also have to be supported by a vertical prop at the crown of the arch and a horizontal prop against the feet of the steel arch ribs founded on rock or mass concrete during excavation and concreting.

Pipe Ramming and Temporary Support

The primary temporary support for the tunnel excavation and construction is the use of installation of the horizontal pipes to form a temporary arch support around the perimeter of the tunnel (Fig. 5). The technique is to use a pipe ramming machine able to push 600 to 800 mm diameter steel pipes in a straight line into the ground. The ramming plant primarily includes the ramming equipment (called "GRANDORAM" made by Grundoram Koloss in Germany), air compressors and generators.

The method is used to install interlocked steel pipes around the perimeter of the underpass, independent of the geometry of the underpass cross section. Interlocked longitudinal joints between neighbouring pipes act as a guidance tool during pipe installation, and later on, during the tunnel construction, as a protection against water and soil inflows. The interlocked joints do not have a structural purpose, even though they have some shear capacity in the pipe cross-section direction.

Pipe ramming is generally performed from one side of the underpass with an exception that where the existing box culvert is present, the pipe ramming will be carried out from both sides. At the driving side, a sheet pile wall is installed to fix the face where steel pipes will be rammed toward the body of the embankment. As the tunnel cross section is up to approximately 7 m high, a temporary platform made of steel members is formed to reach the location where upper steel pipes will be rammed. On the platform, steel rails are prepared for the ramming operation. The first segment of the steel pipe will have a "cutting nose" placed on the head of a steel sleeve. The ramming hammer on the rear part of the pipe will push the pipe through the hole cut in the sheet pile wall toward the embankment, and subsequent steel pipe segments will be welded on for the next push. Vibration of the steel pipe is to be expected on the rear part of the steel pipe, not on the front. The soil remains within the steel pipe installed.

The method does not leave a gap between the exterior of steel pipe and the surrounding soil, and therefore negligible settlement would be anticipated during ramming. Directional and positional control and guidance are provided by a sensor-based system within the pipe, if necessary. Correction is possible by moving the rear part of the pipe.

Given the limited thickness of the overburden soil to the road pavement, seven 600 mm diameter pipes are used in the crown area of the tunnel. For the remaining portion, 800 mm diameter pipes will be installed. After all pipes (total 31) around the design geometry of the tunnel are installed, the temporary support is essentially established. Excavation of the tunnel cross section on the face can then start after the steel ribs are installed at the portal outside the sheet pile wall.

Face Soil Treatment and Excavation Requirements

The soil inside the arch is to be treated prior to commencement of the excavation works for the tunnel, to maintain the face stability. This treatment is required to ensure a steep temporary cut slope for the tunnel excavation advancing along the alignment. For a target factor of safety not less than 1.2 for the temporary cut slope of 60 degrees, the soil is required to have a strength of c'= 17 kPa and $\phi' = 35^o$. The improvement should be relatively easy to achieve by cement grouting. Alternatively, other treatment methods, such as temporary steal bars or nails installed and grouted into the face, can also be used, prior to each round of excavation. These should also be at least double the length of one round to ensure an overlap. They have to be removed as the face is excavated.

Excavation steps should be limited to 1.0 to 2.0 m advance length. After each excavation step, the temporary support will be applied in the form of steel ribs with shims immediately against the pipes arch. Temporary support measures will enable concreting of the final lining using steel formwork in a length of about 5.5 to 7.5 m.

The excavation has been designed to have one advancement face over the entire cross section. A typical detail of the construction procedures is shown on Fig. 4. After opening the cross section through the starting sheet-pile wall, the excavation starts from the top to the bottom down to either the rock level or the design formation level, whichever is deeper. A shotcrete layer will be placed between the pipes to form a "smooth" surface, and the supporting steel rib can then be placed on either bedrock or mass concrete over bedrock. The double ribs (frames) below the Tuen Mun Road carriageway are designed to be placed at 6 m intervals except at the portal ends which are at 7.5 m, and be supported by a vertical prop at the crown extending to the bedrock, as well as a horizontal prop at the base of the steel ribs.

It is noted that the actual groundwater level within the excavation area would be around 2.5 to 4 mPD based on the piezometer measurement at the site. The excavation level is approximately at 3.5 mPD and, therefore, groundwater flow into the excavation is likely of less concern during construction.

DESIGN PARAMETERS

Geotechnical parameters used in the analysis are based on the site investigation data and results obtained from the corresponding in situ and laboratory tests. The design parameters related to the soil-structural interaction analysis are summarized in Table 1.

Table 1: Geotechnical Parameters Used in FLAC Analysis

Material	Unit weight γ_t (kN/m^3)	Young's modulus E_s (MPa)	Cohesion c' (kPa)	Angle of internal friction ϕ' (°)
Embankment fill	20	35	0	35
Alluvial sand	20	20	0	35
CDG	21	35	5	35
Bedrock	25	5,000	100	45

The Young's Modulus (deformation modulus) E_s of the soils is derived from the empirical correlation with the field measured penetration resistance N values (blows/300mm) from the Standard Penetration Test (SPT). For soil materials, $E_s = c\ N$ (MPa) where c is a correlation coefficient. A range of c values is reported. Plumbridge et al (2000) reports c = 3 to 6 based on back-analysis of full-scale pile load tests. Davies (1987) reports c = 1.4, which is used for completely decomposed rock in Hong Kong by some designers. Whiteside (1986) indicates c = 0.2 to 2 for granite saprolite. In general the value of the coefficient c ranges from 0.2 to 6. The higher range corresponds to higher pre-loaded, compacted or dense materials. For the FLAC modeling presented herein, a c = 2.3, 2 and 1 is adopted for the embankment fill, alluvial sand and CDG material, respectively. The adopted design values of Young's moduli are within the typical range for soils in Hong Kong (GEO, 1993).

A Young's modulus (E) of 200 GPa is used in the analysis for the steel ribs or props. An elastic modulus of 26 GPa is utilised for the concrete used for the permanent lining in the modelling. For design purposes, ground water level is taken at 8 mPD as required in the Particular Specifications for the Contract of the tunnel design.

NUMERICAL ANALYSIS

The excavation and shoring support for the tunnel construction of the proposed underpass is modelled and analysed using the computer programme FLAC (Itasca Consulting Group, 1995). The appropriate construction activities, including excavation, temporary support, and installation of permanent lining, were considered in the analysis.

Analytical Model

A critical section near the middle of the tunnel alignment (Fig. 2) is selected for the numerical analysis of a two-dimensional problem. A generalised geological profile was used for the modeling.

The discretization of the continuum and the displacement boundary conditions were modelled to accommodate both the geometry of the tunnel and the construction sequence. The mesh of the model is shown in Fig. 6. The main temporary steel ribs and props, and the permanent concrete lining have been modelled as a series of elastic beam elements. In the numerical model, the temporary intermediate steel ribs is conservatively ignored since they will be removed after installation of the main ribs. The individual strength or stiffness of the horizontal pipes is not considered, since the pipes will transfer the overburden pressure to the main double steel ribs and the interlocking between pipes is provided as an extra protection mainly for the purpose of prevention of loss of ground and water leakage.

The vertical boundaries of the model were set a sufficient distance from the excavation region (generally on the order of 5 to 6 times of the excavation depth) to effectively eliminate boundary effects. The lower horizontal boundary was set at -8.5 mPD well below the bedrock surface. The two vertical boundaries and the lower horizontal boundary are restrained in both horizontal and vertical directions. Small model elements were used near the excavation region where changes in strain and stress are expected to be more significant and of primary concern.

The Mohr-Coulomb constituent model was used for soil and rock in the analyses. In the FLAC modeling, no water drawdown was considered outside the excavation since a contiguous interlocked pipe arch and permanent lining with waterproof membrane are to be installed along the tunnel. This is conservative as it allows full water pressure acting on the structure.

Modelling of Construction Sequence

The construction of the tunnel will be carried out using a top-down construction technique advanced from the north driving end. The construction sequences were modelled as closely as possible in FLAC analyses for two dimensional problems. The appropriate construction activities were grouped into stages, starting from setting up the initial ground conditions to the final excavation level. The construction stages, after initial ground conditions and installation of the pipe arch, are generally modelled as follows:

Stage 1 Dewater and excavate to form the design tunnel void and install the main steel ribs, as well as vertical props from the crown of the arch to bedrock and horizontal prop connecting the feet of the steel arch ribs (construction stage).

Stage 2 Install permanent concrete lining including the tunnel base slab for the road, and remove "all" temporary steel ribs and prop supports (permanent conditions).

Analytical Results and Discussions

The predicted ground movements and structural forces are presented for both model stages. It is found that the most critical stage, in which maximum ground movement and structural forces were computed, occurs at completion of construction works (permanent conditions after installation of lining). A summary of the results computed from FLAC is given in Table 2 below. The table summarises the computed maximum structural forces at different stages and maximum accumulative ground movements.

Table 2: Maximum Structural Force & Ground Displacement Com puted from FLAC

Model Stage	Description	Structural force (moment: kN.m/m; force: kN/m)			Accumulative surface settlement at crown (mm)
		Bend. moment	Axial force	Shear	
1	Steel rib	198	533	166	4.3
	Vertical prop	N/A	404	N/A	
	Horizontal prop	N/A	33	N/A	
2	Permanent lining	350	821	293	19.4
	Base road slab	444	250	218	

It is seen (Fig. 7) that during stage 1, the ground surface settlement at the crown location is predicted to be 4 mm, which essentially results from the deflection of the steel ribs. After removal of the temporary steel ribs, it is predicted that a further 15 mm ground settlement at the crown location will take place due to deflection of the permanent lining. The analysis shows that the ground settlement generally reduces with increasing distance from the crown of the arch (Figures 7 and 8). Ground movements on both sides of the tunnel are approximately symmetrical around the centre line of the tunnel. Figure 8 shows the ground movement pattern after permanent lining is installed and temporary support removed. The plot indicates that the ground at the tunnel crown area would move downwards and the lower two-third portion of the tunnel on both sides displace outwards. The prediction suggests that the ground may be subject to heave of 1 to 2 mm at about 10 to 15 m distance from the centre line of the tunnel.

The tunnel will be essentially built within the embankment fill material of Tuen Mun Road. Sensitivity studies with respect to the deformation modulus of the embankment fill have been conducted by reducing the modulus by 50% (i.e. E_s = 18 MPa, c = 1.2) in the FLAC modelling. The results show that a reduction of the modulus by 50% would lead to an increase in maximum ground settlement by 7% at the end of the construction. Analyses were also carried out to study the sensitivity to the structural stiffness and arrangement of temporary support in order to optimise the design. With consideration of the potential risk and practicality, the adopted final design is such that the grouting need outside the lining is eliminated with introduction of a stiffer temporary structural support.

Since the longitudinal horizontal pipes are not modelled in the FLAC analysis, a separate calculation using the computer programme SuperSTRESS (Integer 1995) is performed to estimate the downward deflection of the pipe at the crown location. The maximum deflection of the top pipe under the carriageway supported by the main steel ribs during temporary stage is predicted to be 0.4 to 0.7 mm. The impact of the above pipe deflection on the ground surface settlement is expected to be negligible. The total accumulative maximum settlement on the Tuen Mun Road surface is estimated to be of the order of 20 mm, which is predicted to satisfy the design criterion of maximum 20 mm movement.

Discussions on Construction Progress

All the horizontal pipes forming the temporary arch were installed. The temporary double steel ribs have been all placed. The excavation works were carried out from both ends of the tunnel to expedite the construction programme and was completed. The survey of the ground surface settlement markers indicates that ground surface settlement of 6 mm at the crown location has occurred during stage 1. This compares quite well with the predicted value of about 5 mm total. The measurement in the horizontal inclinometer installed along the alignment of the tunnel at the crown shows that the pipe (at the crown) deflection is negligible (less than 1 mm).

Conclusion Remarks

An arch shape structural lining is adopted to replace the original rectangular box tunnel structure. Trial grouting prior to construction showed that an effective grouting treatment within the 1.5 m overburden between the tunnel roof to the Tuen Mun Road surface is very difficult to achieve, because of the allowable low grout pressure. An optimised design by using a stiffer structural support has been adopted and the requirement for pressure grouting to treat the soil outside the permanent lining is eliminated. So far the construction of the temporary support works – the pipe arch and steel ribs have been successfully completed. Casting of the permanent lining and other permanent works are under way. The tunnel works is expected to be completed by June 2002.

References

Davies J.A. (1987). "Groundwater Control in the Design and Construction of a Deep Excavation". Proc. of the 9th European Conference on Soil Mechanics and Foundation Engineering, Dublin.

GEO (Geotechnical Engineering Office). (1993). *Guide to Retaining Wall Design (Geoguide 1),* Geotechnical Engineering Office, Hong Kong Government.

Integer. (1995). *SuperSTRESS User Manual, ver 3.1*, Surrey, UK

Itasca Consulting Group. (1997). *FLAC ver3.3 User Manual*, Miniapolis, USA

Plumbridge G.D., Sze J.W.C. and Tham T.T.F. (2000). "Full Scale Lateral Load Tests on Bored Piles and a Barrette". Proc. of the 19th Annual Seminar of the Geotechnical Division of the HKIE, 12 May. pp213 – 220.

Whiteside P.G.D. (1986). "Horizontal Plate Loading Tests in Completely Decomposed Granite". Hong Kong Engineer, Vol. 14, No. 10, pp7-14.

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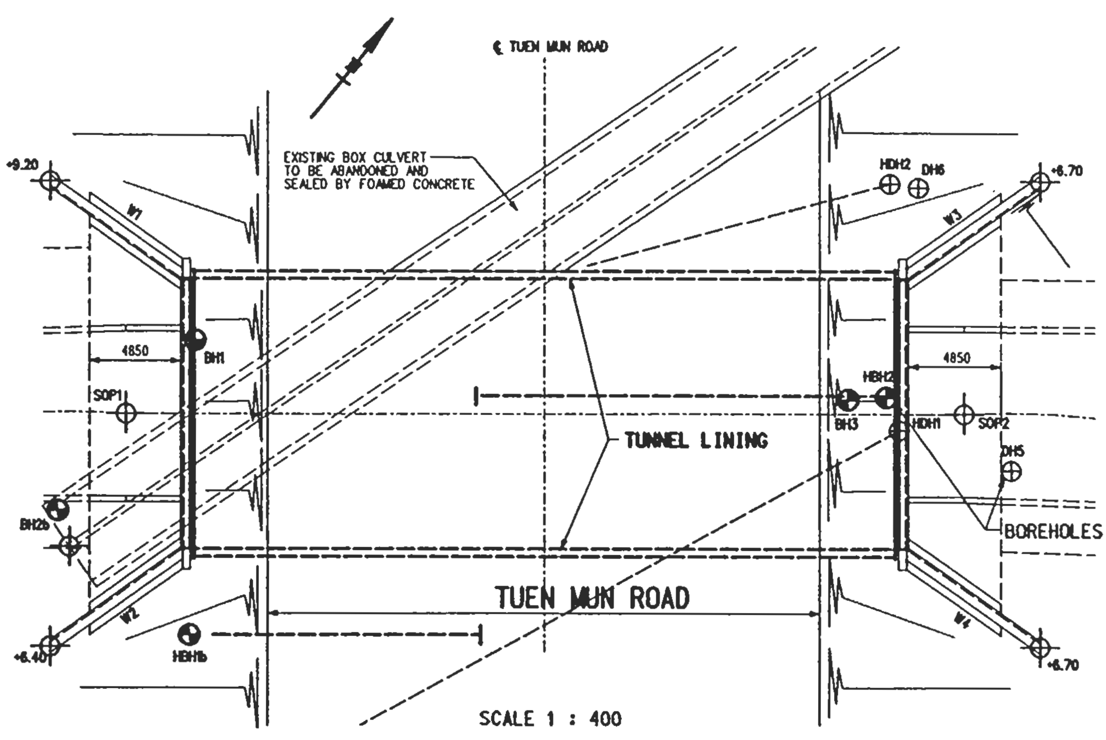


Figure 1 : Site Plan

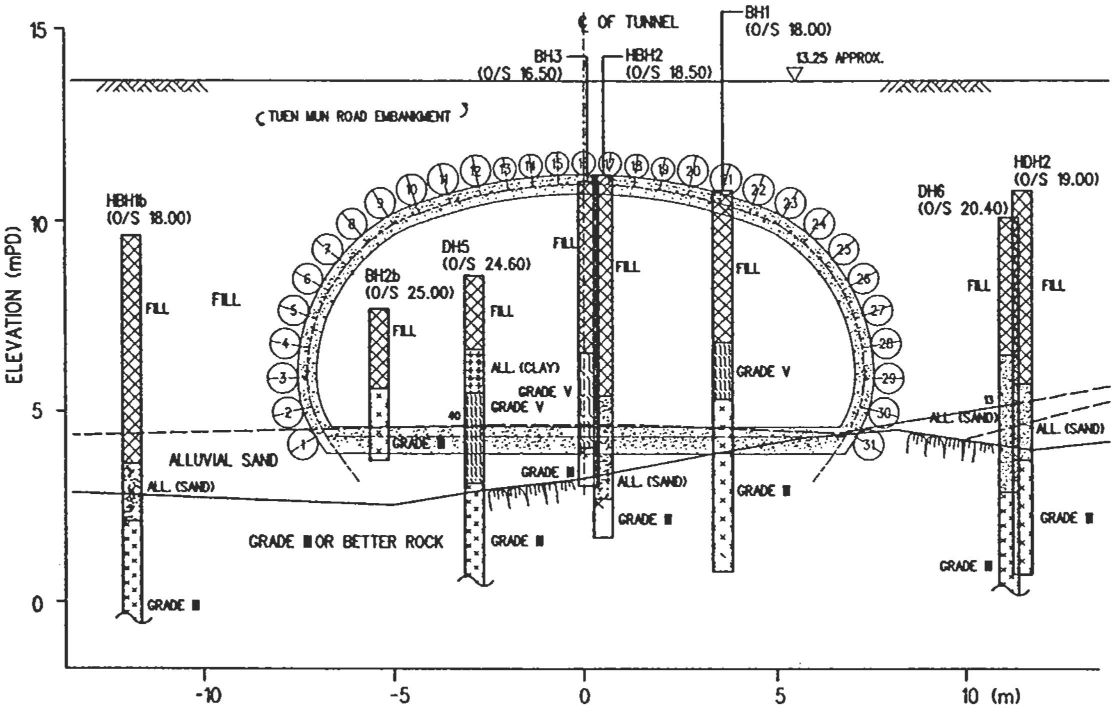
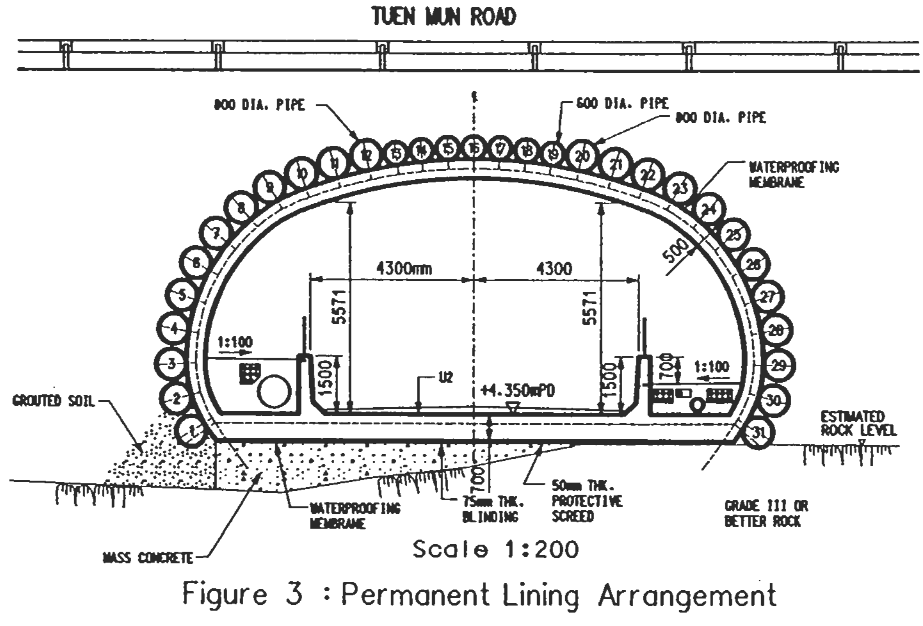


Figure 2 : Typical Geological Cross Section



Scale 1:200

Figure 3 : Permanent Lining Arrangement

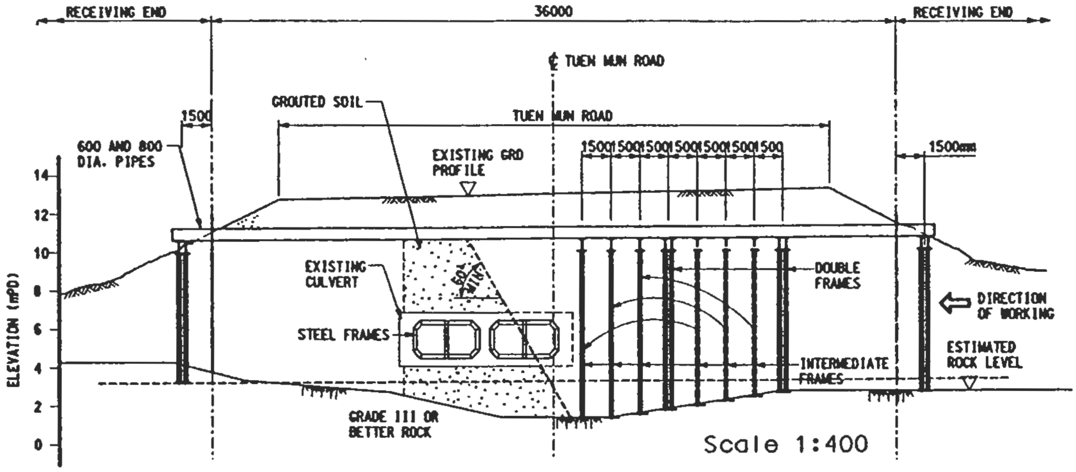


Figure 4 : Sequence of Works and Support

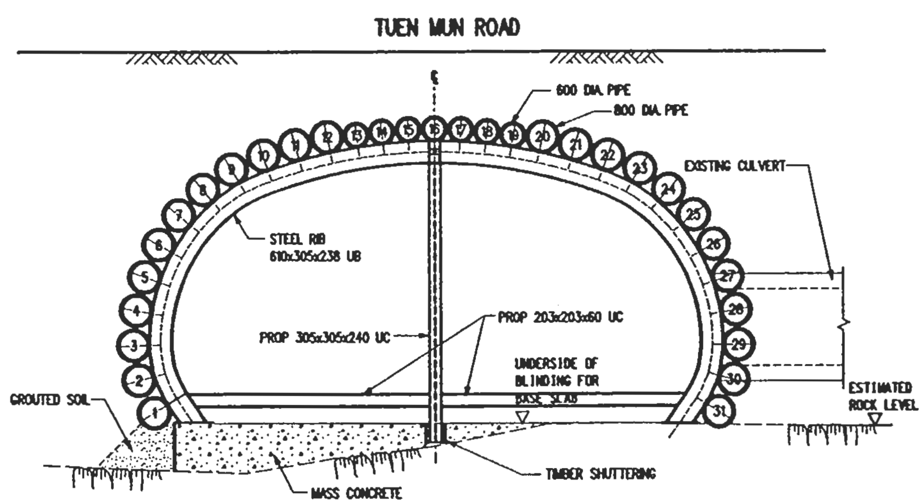


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Figure 5 : Temporary Support

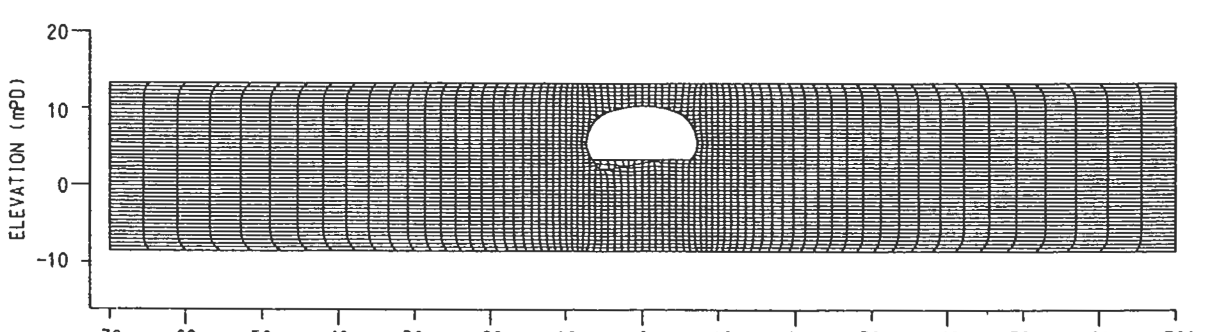

Figure 6 : Mesh of FLAC Model with Tunnel Installed

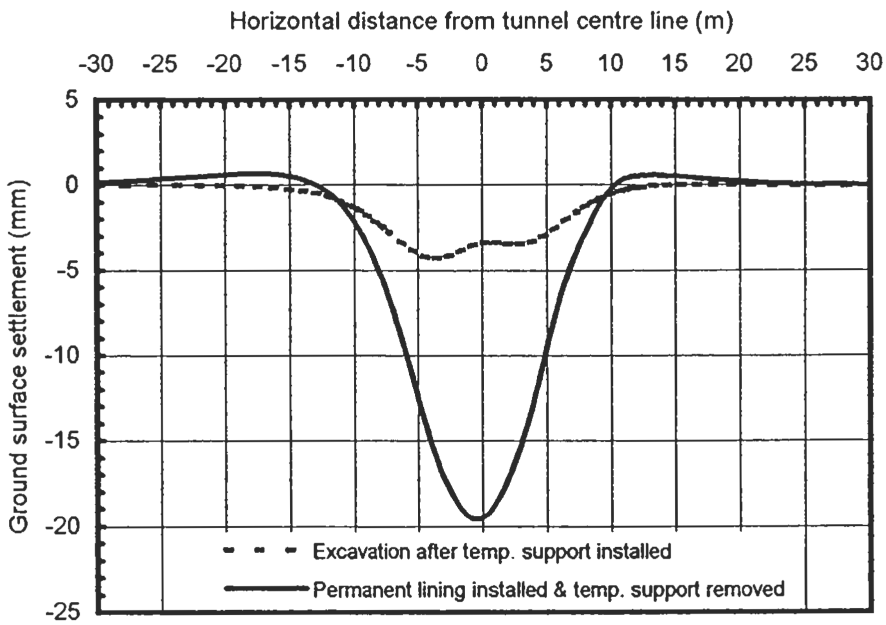

Figure 7: Predicted Ground Settlement Profile

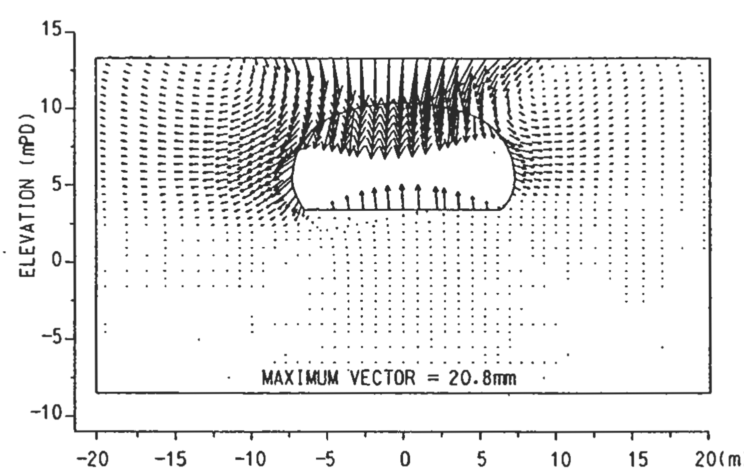

Figure 8 : Ground Movement Vectors

青衣北岸公路的美觀土力設計

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土木工程署土力工程處

AESTHETIC GEOTECHNICAL DESIGN FOR THE TSING YI NORTH COASTAL ROAD PROJECT

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撮要

青衣北岸公路位於青衣島北岸的丘陵地帶，全長 2.2 公里，連接大嶼山通道到香港機場和大嶼山其他地方。這項土力工程包括了斜坡、擋土牆和天然滑坡護土設施的建造。爲了改善這等工程對景觀和環境的影響，所有人造地形均被盡量設計到融合於現存的景物，以達到美觀效果，本文詳述了這等工程的設計。

AESTHETIC GEOTECHNICAL DESIGN FOR THE TSING YI NORTH COASTAL ROAD PROJECT

Anthony Y.T. Lam[1], Kenneth W.K. Lau[1] and K.P. Yim[1]

ABSTRACT: The Tsing Yi North Coastal Road project involves the construction of a 2.2 km highway, which connects with the Lantau Link to form part of the principal access to the airport and other developments on Lantau Island. The highway runs along the northern foothills of Tsing Yi Island, and one of the critical aspects of the project is to minimize the visual and environmental impacts of the geotechnical works for the highway. The ultimate goal is to blend the man-made landforms with the surrounding landscape to achieve visual harmony. Aesthetic aspects relating to the design of slope works, earth-retaining structures and mitigation measures against natural terrain hazards are described in the paper.

INTRODUCTION

The Tsing Yi North Coastal Road is located along the northern coastline of Tsing Yi Island (Figure 1). The layout of the project is shown in Figure 2. About two-third of the highway is supported on viaducts with the remainder in cut and fill. The formation of the cut and fill highway sections required geotechnical works on a major scale along the northern foothills of Tsing Yi Island, and included:

- Soil and rock cut-slopes ranging from 20 m to 70 m in height;
- Reinforced fill embankments up to 15m in height;
- Earth retaining structures ranging from 8 m to 16 m in height founded on sloping ground;
- Protective barriers up to 3m in height for the mitigation of natural terrain hazards.

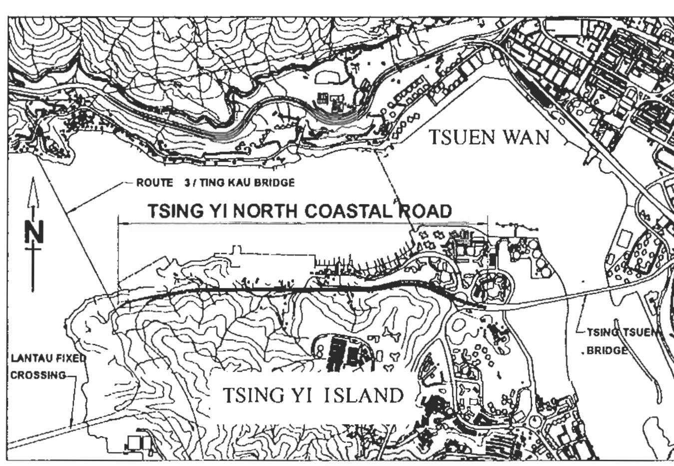


Figure 1 - Location plan.

In order to minimize disturbance to the natural terrain, aesthetic elements were integrated into the geotechnical design to facilitate the blending of the man-made slopes, earth retaining structures and natural terrain hazard mitigation works with the surrounding natural hillsides. Special attention was paid to the visual appearance of the final landform and the sustainability of the aesthetic design.

The contract sum for the civil engineering works is around 775 million Hong Kong Dollars. Construction of the project commenced in February 1999 and the road was opened on 2nd February 2002.

[1] Geotechnical Engineering Office, Civil Engineering Department, Government of the Hong Kong Special Administrative Region.

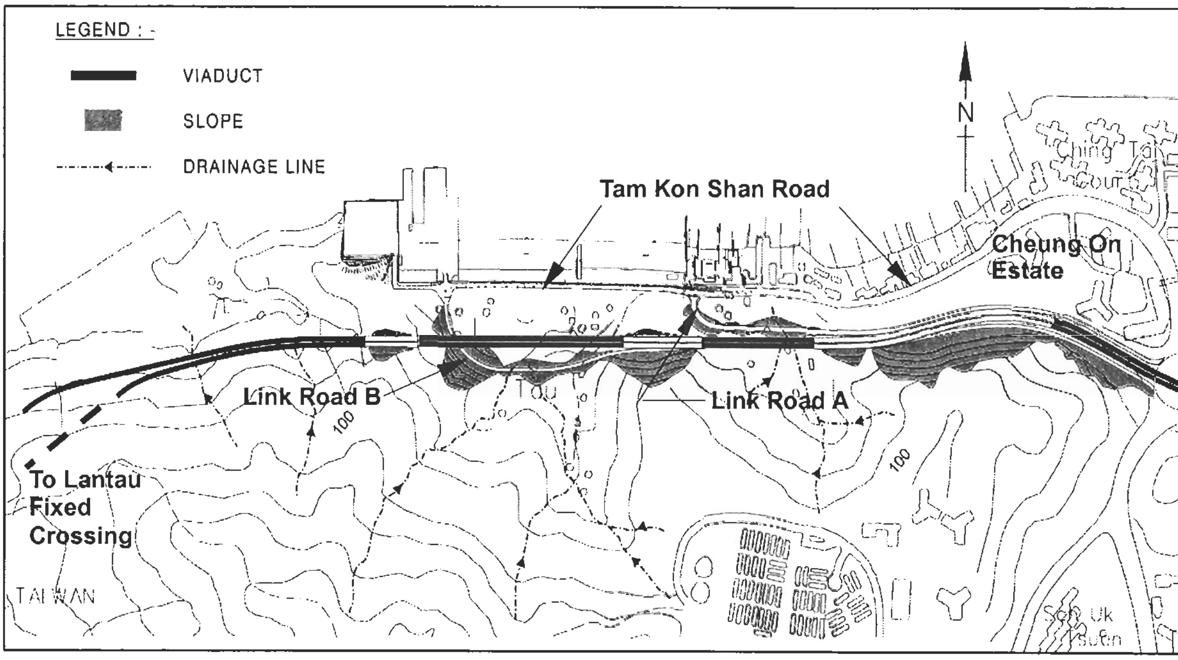


Figure 2 - Project layout plan.

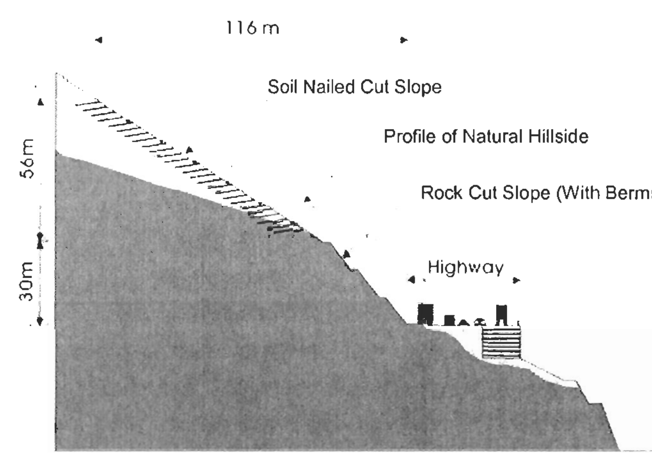
AESTHETIC GEOTECHNICAL DESIGN

The concept of aesthetic geotechnical design is to blend the man-made landform with the surrounding landscape to achieve visual harmony. Aesthetic aspects relating to the design of slope works, earth retaining structures and mitigation measures against natural terrain hazards of this project are described below.

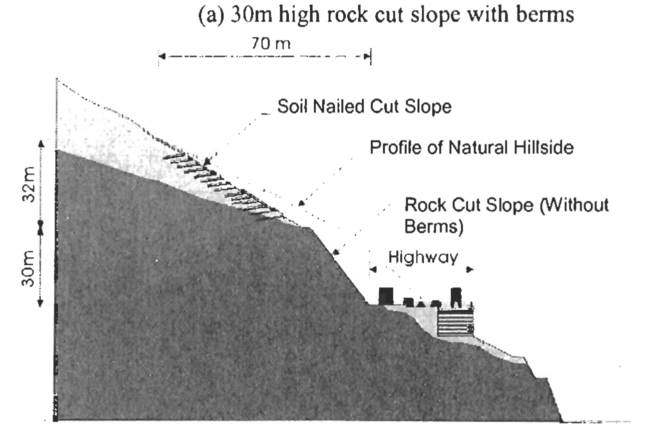
Slope Works

One of the prime objectives of aesthetic slope design for large-scale site formation works is to minimize disturbance to the natural hillsides and at the same time softens and screens the visual impact of the man-made slopes. In this project, the omission of berms on high rock cut slopes, as illustrated in Figure 3, could drastically reduce land take and excavation, hence, minimizing disturbance to the natural terrain.

As blending the man-made landform with the surrounding natural terrain requires successful establishment of vegetation on the cut and fill slopes, close attention to construction detailing has been found to be very important for aesthetic slope design. The use of erosion control mat to facilitate the long-term enhancement of vegetation stability and also to reduce visual prominence of the man-made features, such as soil nail



(a) 30m high rock cut slope with berms



(b) Omission of rock berms reduces excavation, slope stabilization works and disturbance to natural hillside

Figure 3 - The effects of rock berms on cut slope design (Lam & Yim 2001)

heads on slope surface, has been successfully adopted in this project. As protruded nail heads can prevent the retention of topsoil within the erosion control mat during heavy rainstorm, all soil nail heads in this project are recessed from the slope face to allow uniform spreading of topsoil on the slope (Figure 4). Another important construction detailing that contributes to the successful retention of topsoil on the slope is the proper installation of the erosion control mats. For effective performance, sufficient anchorage points are required to fix the mats close to the undulating slope surfaces. Although the relatively thin layer of topsoil retained inside the erosion control mat directly above the soil nail heads is unable to allow the growth of trees or shrubs, the establishment of grass on the slope surface has successfully concealed the nail heads (Figure 5).

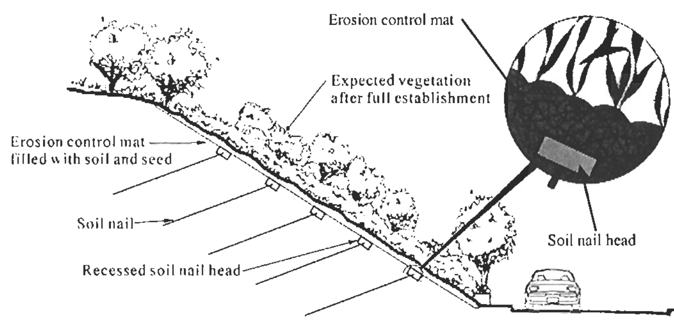


Figure 4 – Illustration of recessed soil nail heads and erosion control mat

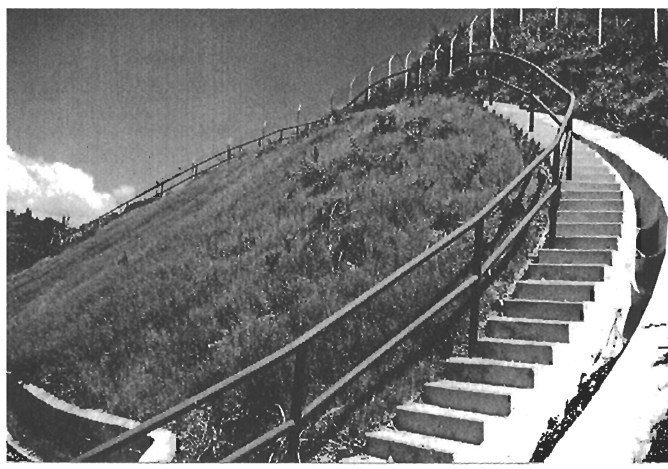


Figure 5 – Establishment of vegetation on a soil nailed slope with the protection of erosion control mat

Native vegetation species are under-represented in current hydro-seeding and transplanting practice. Some of the foreign species tend to develop a thick canopy on the ground to inhibit healthy undergrowth. In order to maintain a balanced ecological environment, various vegetation species indigenous to southern China were used in the project. In addition to hydroseeding the slope surfaces, shrubs with a shallow rooting system were planted on the slope together with trees planted along the slope toe to introduce a continuous linear corridor of planting along the new highway.

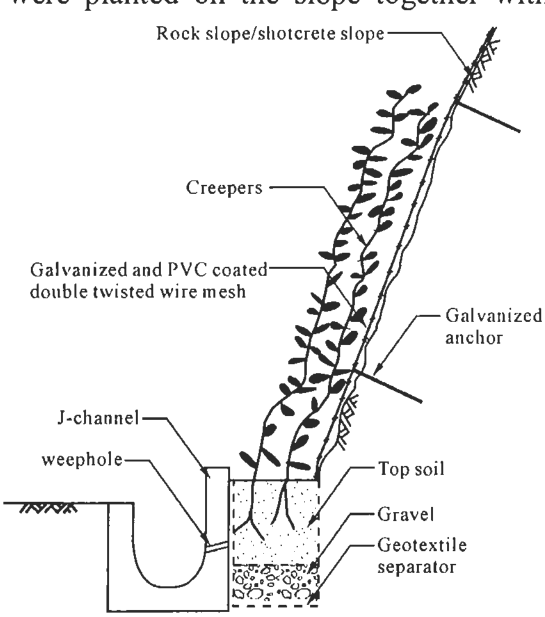


Figure 6 - Planting details for rock slopes and steep slopes with shotcrete surface

Several techniques for improving the appearance of rock slopes have been adopted in the project. In order to minimize the visually intrusive rock slope stabilzation works including shotcreting, wire netting has been provided on the surface of rock slopes to prevent dislodgement of rock wedges. Netting tends to allow the natural colour and structure of the rock to be seen and it would also facilitate the establishment of creepers on the slope. The wire nets are securely anchored into the rock face and are protected against corrosion by galvanizing and PVC coating. The typical arrangement of planter, slope toe drainage and netting is shown in Figure 6.

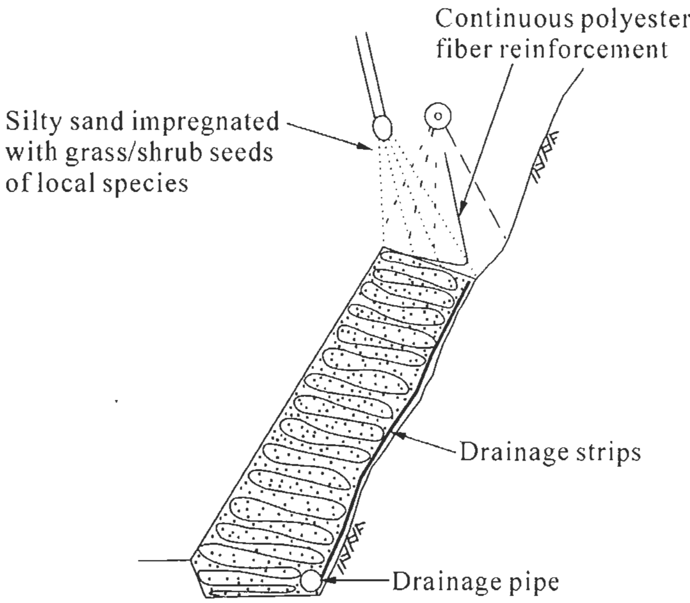


Figure 7 – A 200 mm soil cover reinforced by impregnated polyester fibers

Figure 8 –Establishment of vegetation on a 20m high rock slope

Normally creepers with successful trial records in Hong Kong could take longer than 2 or 3 years, depending on the degree of horticultural care, such as periodic watering during the initial growth period to fully established on the slope face. A pilot vegetation trial that employs the technique of fiber-reinforced soil has been implemented on a 20 m high rock slope along the main carriageway. The fiber-reinforced soil method has been applied successfully in Japan in the past decade. It employs spraying a mixture of continuous fibers and silty sand with seedlings of local species to form a 200 mm thick soil cover on the rock face or shotcreted surface. The resulting 200 mm soil cover is basically a self-supportive soil composite reinforced by the impregnated polyester fibers. Internal drainage behind the soil layer is provided by a network of polymeric strip drains. The typical arrangement of the fiber reinforced soil technique is shown in Figure 7. The long-term stability of the soil cover will be further enhanced with the establishment of the roots of the vegetation cover. Growth of vegetation on the 20 m high rock slope shortly after hydroseeding is shown in Figure 8. Long term monitoring has been included in the trial to assess the effectiveness of this quick greening technique.

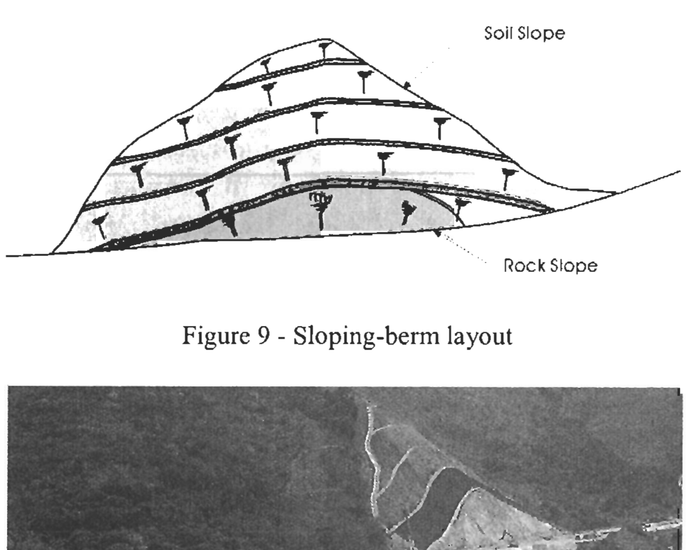


Figure 9 - Sloping-berm layout

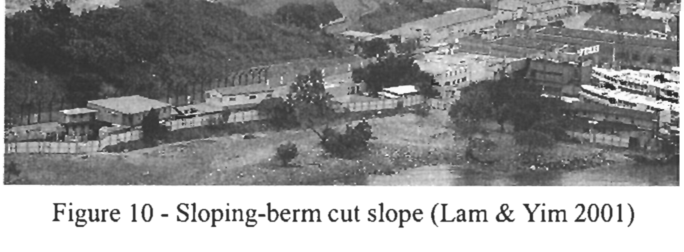


Figure 10 - Sloping-berm cut slope (Lam & Yim 2001)

In order to provide slope face contouring to avoid large monotonous "planar" slopes, sloping berms have been constructed for all the cut slopes under this project to give a more natural appearance. Sloping-berm design tends to fit with the natural topography better, and it usually results in a smooth transition between the natural terrain and the man-made landform and a reduction of the total volume of excavation in comparison with the conventional level-berm design. Unlike the conventional level-berm design which needs step-channels and down-pipes at intervals on the slope face to collect water from U-channels on berms, the provision of sloping berms across slope face at an average gradient of V:H = 1:6 will enable the drainage channel to be self-cleansing, hence, minimizing the provision of visually intrusive step channels and down-pipes on the slope face and long-term maintenance of the slope drainage. An illustration of the sloping-berm arrangement is given in Figures 9 and 10.

Figure 11 – A 16 m high reinforced fill wall under construction (Lam et al 2001)

Earth Retaining Structures

In order to soften the visual impact of the earth retaining structures that were constructed on the hilly terrain to support the new highway, aesthetic elements have been incorporated in the design. Among the various retaining walls supporting the highway, the largest is a 200 m long reinforced fill wall spanning across a valley, with a maximum height of 16m at the valley floor (Figure 11). The wall is located immediately above the cut slopes of an existing road, which is 25 m below and runs parallel to the new highway.

Formed of prefabricated facing panels, the reinforced fill wall allows greater flexibility in integrating aesthetic elements to achieve innovative and practical aesthetic

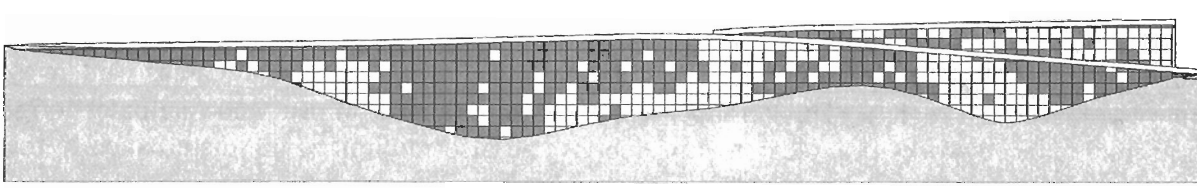


Figure 12 - Architectural finish on the reinforced fill retaining walls

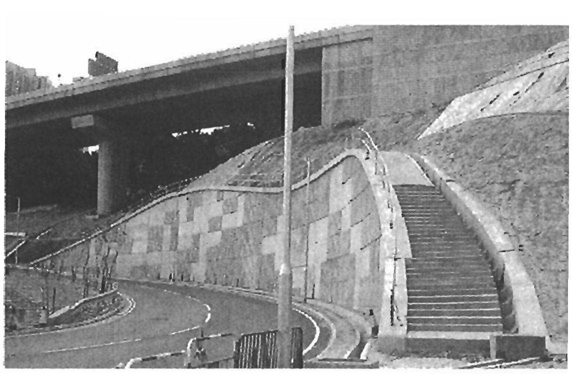


Figure 13 –Architectural finish on a 8m high bored pile wall

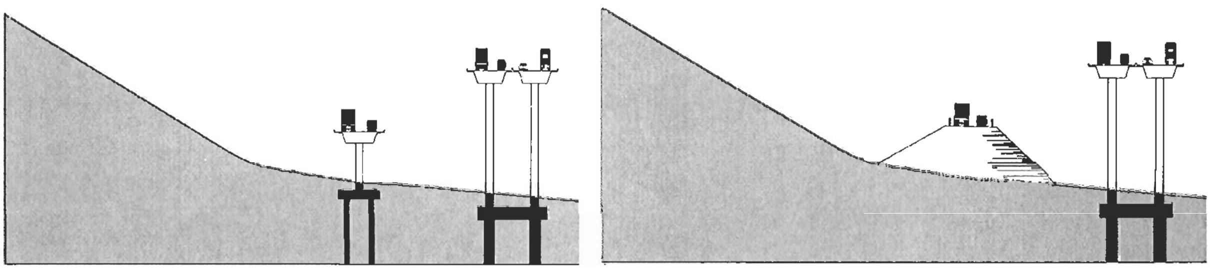
design solution. In this project, the surface of the reinforced fill wall is enhanced by a random combination of smooth and ribbed face pre-cast concrete panels, as illustrated in Figure 12. Other cast-in-situ concrete retaining walls (Figure 13) also utilize this unconventional form of wall finishing to break the monotonous surface finish of the conventional concrete retaining structures.

As the appearance of the concrete

retaining structures will deteriorate with time mainly due to water staining and weathering of the concrete, planters are provided along the toe of all the retaining walls including the bridge abutments to facilitate the growth of creepers on the wall face to blend these man-made structures with the surrounding natural landscape.

Natural Terrain Hazard Mitigation

The initial choice of using viaducts to connect the existing roads to the new highway was abandoned in favour of an embankment solution. As illustrated in Figure 14, the viaduct scheme is more vulnerable to natural terrain hazards, which include debris flows and boulder



(a) Viaduct option (more vulnerable to natural terrain hazards)

(b) Embankment option (utilize as barrier against natural terrain hazards)

Figure 14 - A Comparison of viaduct and embankment options (Lam & Yim 2001)

falls. By contrast, the embankment can act as a barrier against the potential natural terrain hazards. As the construction of the embankments utilized excavated materials from the adjoining site formation works, the amount of construction waste that need to be taken off site was reduced.

Apart from the engineering and sustainability functions, the road embankments also help to blend the highway with the surrounding natural landform better. With the viaduct scheme, the columns and the bridge deck will have a major visual impact on the existing environment. On the contrary, the road embankments will conceal some parts of the bridge deck and columns of the adjoining main carriageway, thus help reducing the visual impact of the highway.

Furthermore, natural terrain hazard mitigation measures such as protective gabion barriers against debris flows and boulder fall are provided within the lowland areas well away from the hinterland region. This arrangement reduces the visual impact and facilitates long-term maintenance works (i.e. clearance of debris) and also minimizes the disturbance to the surrounding natural terrain.

CONCLUSIONS

In order to minimize disturbance to the northern foothills of the Tsing Yi Island, aesthetic elements were integrated into the design of the Tsing Yi North Coastal Road to facilitate the blending of man-made slopes, earth retaining structures and natural terrain hazard mitigation works with the surrounding natural landscape. The main objective of the aesthetic design is to make the man-made landforms look as natural as possible and at the same time provides an aesthetically pleasing environment for the local inhabitants and road users.

One important consideration in the geotechnical design is to minimize disturbance to the natural hillsides. The omission of berms on high rock cut slopes has drastically reduced land take and the overall slope height, hence, minimizing the disturbance to the natural terrain.

For the slope design, consideration was given to the slope face contouring to avoid large monotonous "planar" slopes, and at the same time gives a more natural appearance. The adoption of the sloping-berm design for all the cut slopes under this project has resulted in smoother transition between the natural terrain and the man-made landform and a reduction of the total volume of excavation in comparison with the conventional level-berm design. In addition, sloping berm design enables the drainage channels to be self-cleansing, hence, minimizing the provision of visually intrusive step channels and down-pipes on the slope face and long-term maintenance of the slope drainage.

As grassed surfaces are not in themselves mature forms of vegetation in Hong Kong's sub-tropical environment, initial hydroseeding on slopes was followed up with the planting of small seedling trees and shrubs. Visually intrusive rock slope stabilization works including shotcreting were reduced to a minimum by the provision of wire netting on the rock face to prevent dislodgement of potentially unstable rock wedges. Netting tends to allow the natural colour and structure of the rock to be seen and it would also facilitate the establishment of creepers on the rock slopes.

A pilot vegetation trial that employs the technique of fiber-reinforced soil has been implemented on a 20m high rock slope along the main carriageway. Long term monitoring is currently being undertaken to assess the effectiveness of quick greening on rock face or shotcreted face using this technique. If successful, it will provide the geotechnical designer an alternative method of slope surface protection for projects at which visual impact is a major concern.

An unconventional random combination of smooth and ribbed face architectural finish was provided to several large earth retaining structures that support the new highway. This form of wall finishing has broken the monotonous surface finish of the conventional concrete retaining walls including the bridge abutments.

The alignment and position of protective barriers for the mitigation of natural terrain hazards such as debris flows and boulder falls have been carefully chosen to minimize their visual impact. As the hazard mitigation measures are provided within the lowland areas well away from the hinterland region, the disturbance to the natural terrain is minimal and at the same time facilitates long-term maintenance works (i.e. clearance of landslide debris).

REFERENCES

Lam, A Y T & Yim, K P (2001). Sustainable Geotechnical Design for the Tsing Yi North Coastal Road Project, Hong Kong. *Proceedings of the 14th Southeast Asian Geotechnical Conference, 10-14 Dec 2001, Hong Kong. Vol. 1, pp 373-378.*

Lam, A Y T, Lau, K.W. K. & Lee, C.K.T. (2001). Design and Construction of a 16m High Reinforced Fill Retaining Wall in Hong Kong. *Proceedings of the 14th Southeast Asian Geotechnical Conference, 10-14 Dec 2001, Hong Kong. Vol. 1, pp 361-366.*

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香港屯門公路擴闊工程成功的主要因素
- 詳細的地盤勘察及創新設計意念

查卡達、菲達卡夫	高達工程加拿大有限公司－安大略分公司
畢彼得	高達工程澳大利亞有限公司
史提夫巴朗	高達工程加拿大有限公司－英屬哥倫比亞分公司
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INTEGRATION OF DETAILED FIELD INVESTIGATIONS AND INNOVATIVE DESIGN – KEY FACTORS TO SUCCESSFUL WIDENING OF THE TUEN MUN HIGHWAY

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撮要

在香港屯門公路的擴闊工程中，集合了大量的地盤勘察、設計及建築上所需的人力物力，原因是爲應付陡直的削峻、廣大範圍的削弱土層地帶及公路上交通流量非常高。在該項工程中，香港首次在公路上利用高能量岩石滾落保護欄杆及新穎的石釘擋土牆，此公路乃是全港的高速公路之中最困難進行直線核對及路面擴闊工程的公路之一。在早期的公路與擴闊工程所涉及的傷亡訴訟結果中，要求整項工程的程序，包括勘察至施工過程亦需要有嚴謹的安全措施。隨之以後，總工程顧問公司聘請了一隊岩石專家成爲該項工程的顧問。此文章重點集中在土力工程設計及公路旁的岩石削破穩固性的改進發展，並勾劃出多類型可行的岩石工程方案，用作進行安全的公路擴闊工程。

INTEGRATION OF DETAILED FIELD INVESTIGATIONS AND INNOVATIVE DESIGN – KEY FACTORS TO SUCCESSFUL WIDENING OF THE TUEN MUN HIGHWAY

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ABSTRACT: Detailed geotechnical investigations, facilitated the design of innovative solutions to tough constructability issues associated with the required widening of the Tuen Mun Highway. The application, for the first time in Hong Kong, of high energy rockfall protection fences and novel tie-back retaining wall design arrangements contributed to the successful completion of one of the most difficult highway realignment and widening programmes ever undertaken in the Territory. In view of the difficulties in developing a workable solution, Golder Associates was retained as part of the civil consultant group following the Mediation proceedings related to a fatality in an earlier attempt at highway widening. This paper concentrates on aspects of the geotechnical design solutions developed to improve the stability of the existing rock cuts adjacent to the highway, and outlines the various workable rock-engineering solutions developed to achieve the difficult task of safely widening the carriageways.

INTRODUCTION

Background

The Tuen Mun Highway, initially constructed between 1974 and 1983, was Hong Kong's first high capacity expressway, and still remains one of the busiest commuter links between the burgeoning New Territories settlements, the neighbouring Chinese Guangdong Province and Hong Kong Island. This highway has seen some of the regions' worst ever traffic congestion. In order to alleviate this congestion, a highway realignment and widening programme was selected as the most appropriate solution for increasing capacity. In the late 1980's the need to provide climbing lanes and hard shoulders through the uphill sections was established as a key component of any traffic alleviation scheme. Initial construction contracts to achieve widening of the Kowloon bound carriageway, were tendered in 1992. The Highways Department awarded the works under a Design and Build Contract, with initial construction activities starting in 1994. However, in August 1995, a rockfall occurred resulting in a motorist fatality. Following this incident the Contractor claimed the works for the Tai Lam Section were impossible to execute in accordance with the contract, and a subsequent mediation of the claim found in favour of the Contractor. The Coroner, in evaluating the cause of the accident, specifically advised that prior to beginning any further excavation work on potentially unstable slopes (including those at Tai Lam), the areas be thoroughly mapped and analysed, and that if any negligence in execution of these tasks leading to further incidents were proven, the designers could be held criminally responsible.

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The Project

In view of the fatality that occurred in 1995 and the difficulties that had already occurred in developing a workable solution, Highways Department retained Maunsell Consultants Asia Ltd. as the main consultant for the remaining works, who in turn sub-contracted Golder Associates (Golder) as the specialist geotechnical consultant. Golder was engaged to (i) examine the stability of the existing and proposed rock slope cuts adjacent to the highway and (ii) to develop workable rock-engineering solutions for the difficult task of safely widening the carriageways. The rock engineering design was complicated by:

- the height of the rock cuts;
- the presence of unfavourably oriented "sheeting" joints in the weathered granite;
- the lack of rockfall catchment at the toe of the slopes; and by
- the occurrence of a major weak fault zone that crossed one of the slopes in a sector where high cut-backs were the only viable measure for maintaining the alignment.

In addition, the design had to maintain dual, 3-lane traffic flows, of typically 9,000 vehicles per hour throughout the entire investigation and construction periods, with only occasional lane reductions and off-peak closures permitted. Figure 1 shows the site location and the extents of the four works sections (TL/S1 to TL/S4).

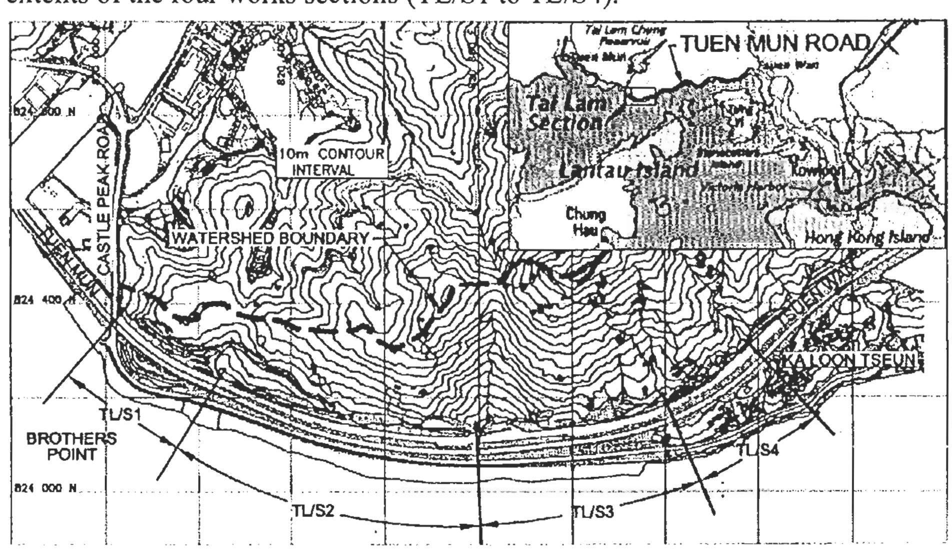


Figure 1: Site locality map, and detail of the Tai Lam Section of the Tuen Mun

Geological Background

The rock cuts along the Tai Lam Section of the Tuen Mun Highway have been excavated within the Lantau Granite, which is locally heavily jointed, with major sheeting joints and fracture zones exhibiting deep penetrative weathering resulting in onion skin slabbing and a large number of corestones and boulders on the natural slopes. The full range of weathering grades is evident on site. Exposed cut slopes consist mainly of moderately to slightly decomposed granite (Grade II/III) with some areas comprising highly to completely decomposed granite (Grade IV/V). In addition to the low angle (25° to 40°) south-west dipping sheeting joint (or stress relief) structures, the rock mass is typically dissected by three other major joint sets. These include a north-west steeply dipping set, a northward dipping (65° to 75°) set, and a third set which dips steeply to the north-east. As the highway trends approximately East-West, but curves round slightly as shown on Figure 1, predominant instabilities are dominated by the outward dipping sheeting structures, with release surfaces created by the other three steeply dipping sets.

Key Issues

Based on evaluations undertaken during the immediate post-Mediation investigations, key slope issues and required new or remedial construction measures were identified for the various rock cuts along the four work sections. These are presented in Table 1.

Table 1: Key slope issues and required works for the Tail Lam Section, Tuen Mun Road.

Tai Lam Section	Key Slope Issues	Required Works
TL/S1	• High, partially unstable rock slope prone to rockfalls • 10 m wide clay-infilled shear/fault zone • Likely stability complications due to high pore water pressures • Boulder zones and segments of potentially unstable natural slopes above the cut slopes	• Undertake extensive excavation to maintain optimum alignment • Cut back the toe of the 50 m high existing rock cut by about 14 m • Create tied-back retaining wall to restrain degraded material within core of shear/fault zone • Carry out boulder stabilisation and install rockfall protection/barriers
TL/S2 & TL/S3	• High, unstable rock slopes prone to rockfalls/slides on sheeting joints • Boulder zones and segments of potentially unstable natural slopes above the cut slopes • Undercut sheeting joint blocks and zones of buttressed sheeting joints with inadequate drainage provisions	• Excavate only a minimal segment at the west end of the TL/S2 cut to achieve required alignment • Carry out substantial rock face stabilisation of existing cut slopes to reinforce sheeting joint structural instabilities (incorporating dowels, buttresses, drape mesh and drainage holes, as necessary) • Carry out boulder stabilisation and install high energy rockfall protection/barriers
TL/S4	• Existing cut slope comprised of completely decomposed granite (CDG) with core stones with spines of rock • Boulder zones and segments of potentially unstable natural slopes above the cut slopes	• Excavate new cut-back into existing shotcreted and supported CDG slope and remove and/or cut core stones appropriately • Install array of soil nails for slope stabilisation • Carry out boulder stabilisation and as required rockfall protection

Design Challenges

As is evident from the above tabulation, two recurring rock mechanics themes were of most importance to achieving the design objectives of successfully widening the east-bound carriageways and adding an extra climbing lane – (a) addressing the excavatability of the two slope segments where new cuts were required, and (b) optimising reinforcement and stabilisation strategies to improve the stability of the existing cuts and natural slope segments that were to remain generally unaltered. Addressing these key issues required developing a detailed understanding of the inter-relationships of major discontinuity trends with slope geometries and compiling relevant data on the shear strength characteristics of the weak discontinuities controlling the stability of the slopes; both major issues that were deemed in the Coroner's hearings and during the Mediation as being significant limitations to previous construction contracts. In order therefore to satisfy the Coroner's riders, and meet the objectives of a constructable design for this difficult and challenging project, key design parameters had first to be adequately quantified.

The key design parameters identified by segment of the slopes are summarised in Table 2.

Table 2: Key design parameters for new, existing and natural slopes.

Slope Zone	Key Design Parameter Requirements
New and Existing Rock Cuts	1. **for Rock Mass Kinematic Sliding Stability Assessment –** • data on orientations and characteristics of the controlling joint sets, • basic shear strength information for the rock joint contact surfaces, • data on large and small scale joint roughness, • information on global rock mass strength, to reflect degree of weathering, as this for the more weathered zones significantly affects overall rock joint shear strength. • information on prevailing groundwater conditions through instrumentation, as previous experience (Slinn and Greig, 1976; Richards and Cowland, 1986) indicated that increases in pore water pressures had dramatic influence on reducing effective shear strengths of the controlling discontinuities, and • key data on transient, peak pore water pressure response behaviour for typhoon conditions. 2. **for Rockfall Trajectory Modelling –** • data on statistical variability in block size distribution of slabs and boulders; and • detailed slope profile and roughness information, to a sufficient accuracy to allow reliable trajectory modelling. 3. **for Soil Slope and Fault/Shear Zone Stability Evaluations –** • determination of geometry of CDG zones, fault zones and extents of fractured weak rock susceptible to ravelling or soil type failure mechanisms, • quantification of CDG, fault and shear zone strength parameters, and • assessment of controlling hydrogeological conditions, through instrumentation.
Natural Slopes	1. **for Assessment of Rockfall Potential & Kinematics of Sheeting Joint Blocks –** • same data as above 2. **for Boulder Stabilisation/Rockfall Mitigation –** • an assessment of the number of boulders, their size, and location distributions, • detailed determination of geometrical characteristics of individual boulders to be stabilised, including; size, shape, depth of embedment etc..., and • slope profile and surface roughness information for trajectory modelling.

INVESTIGATION STRATEGIES FOR RISK MINIMISATION

Acquisition of the data needed for undertaking required stability analyses and completing project scale risk assessments to categorise hazard priorities and define appropriate risk-based safety factors for design, necessitated implementation of a very detailed site investigation (SI) programme. As outlined in more detail in Carter *et al.*, (1998), key objectives of the investigations were

- Assessing the continuity and shear strength of sheeting and other joints, as a basis for design of rock reinforcement and rock slope excavation approaches;
- Determining the transient response of the groundwater system (particularly in the vicinity of the sheeting joints) to heavy, short-duration rainfall events, to allow definition of adequate drainage and pore-pressure assumptions in design;
- Defining the kinematic and statistical variability and controlling block size and joint roughness conditions for better definition of rockfall kinematic parameters, including assessing the size and distribution of boulders in the boulder fields above the cut slopes;
- Defining the detailed ground terrain geometry of the slopes and cuts as a pre-cursor for realistic rockfall trajectory modelling for design of rockfall control measures, and
- Assessing the depth, characteristics and configuration of the CDG zones, core stones in the upper weathered zones and identified fault zones to provide rationale for slope support assessments (retaining structures and/or reinforcement).

Design Measures

Based on the information collected from the site investigations and field assessments, four quite radically different design solutions were formulated to deal with the very different field conditions encountered in various parts of the Tai Lam slopes. -(1) Cut-back rock excavation methods and extensive rock doweling were the approaches deemed most appropriate for dealing with stabilising block slide mechanisms critical to the stability of the new TL/S1 cut and for dealing with sheeting joint slabs high on the existing rock slopes at TL/S2 and TL/S3; -(2) A novel tied-back retaining wall was developed as the most effective approach for stabilising the 10m wide weak fault zone that transected the centre of the TL/S1 slope segment; -(3) High capacity rockfall control fences and draped mesh solutions were the methods chosen for dealing with potential boulder and rock block dislodgement hazards, while -(4) A hybrid conventional soil nail and rock dowel solution was chosen respectively for reinforcement of CDG slope sections with interspersed large core stones and rock spines.

Although there are many individual areas of the overall project site, where specific design and construction measures are noteworthy, the remainder of this paper will concentrate largely on the major works undertaken in the vicinity of the TL/S1 box-cut at the west end of the Project alignment.

TL/S1 - Rock Cut Excavation and Support

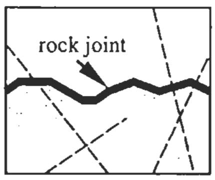
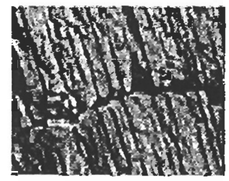
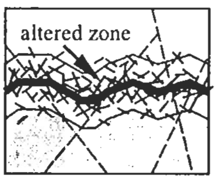
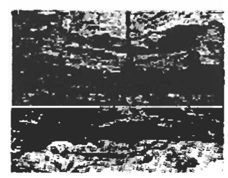
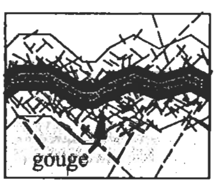
Three significant rock-engineering issues required solution for the TL/S1 slope:

(i) the permanent stabilisation of a major fault zone exposure, of approximately 15m width and some 23m height, with a true thickness of about 10m with a dip of 78° to the north;
(ii) reinforcement of marginally stable major sheeting joint zones and tetrahedral and wedge shaped blocks formed by intersection of prominent sheeting and cross-joints; and
(iii) control of groundwater pressure heads, as data from the existing cut indicated these were unacceptably high for continued stability under adverse conditions.

The new rock cuts required at TL/S1 included removal of the previously existing rock knob on the south side of the carriageway and pushing back the high north side of the box-cut through the ridge. This cut, which is aligned roughly perpendicular to the road, rises to the north from a height of about 15m on the south side of the highway to a height of 65m to the north of the road. On the north side the original cut was about 50m high.

Design of the rock cut geometry for the new set-back required at TL/S1 was generally quite conventional with the exception that wedge and block size geometries of controlling rock blocks was determined by a combination of field mapping and photogrammetric analysis methods as a basis for discrete deterministic block size evaluation using the FRACMAN/ROCKBLOCK computer code (Dershowitz and Carvalho, 1996). Assessment of the stability of the 3D blocks was then undertaken both in 3D using the GOLDPIT computer code and in 2D using conventional limit equilibrium plane and wedge failure analysis methods. Prior to and during these design analyses it became apparent that the key uncertainty relating to support design and optimisation of excavation cut-back temporary support was ensuring an adequate understanding of the continuity and shear strength characteristics of the controlling sheeting joints (which varied from joints with no infill to structures with highly to completely weathered gouge/CDG zones in excess of 300mm in thickness). Although a kalaedoscope of different characteristics could be recognised in the field, in order to simplify design analysis yet realistically ascribe appropriate design strength parameters for the weathered controlling sheeting joints, three basic geological models were defined. These were then characterised by use of two principal constitutive failure criteria, as summarised in Table 3 below.

Table 3: Sheeting and cross joint characteristics and strength criteria.

Strength Control	Typical Geometry	Photograph	Governing Strength Equations
Joint Strength (Rock-Rock)			**Barton-Bandis Criteria for Joints with Rock-Rock contact** $\phi_{(peak)} = JRC.\log_{10}(JCS/\sigma_n) + \phi_r + i$
Wall Rock & Asperity Strength (weathered joint margins, Grade II/III upwards)			**Hoek-Brown Criteria for Degraded Sheet Structures (where weathered wall-rock or gouge material characteristics dominate behaviour)** $\phi = Arc\tan\frac{1}{\sqrt{4h\cos^2\theta - 1}}$
Joint Infill/Gouge (no asperity contact)			where: $h = 1 + \frac{16(m\sigma_n + s\sigma_c)}{3m^2\sigma_c}$ and $\theta = \frac{1}{3}(90 + Arc\tan\frac{1}{\sqrt{m^3 - 1}})$ and m and s are Hoek-Brown constants

The other major uncertainty affecting required design levels of rock support for stabilisation of the various sheeting joint blocks, and also requiring detailed information from the field investigations, was groundwater pressure. Of most critical concern was attempting to define maximum credible design heads and pressure response and decay behaviour of porewater in the sheeting joints under extreme typhoon style, excessively heavy, short duration rainfall events.

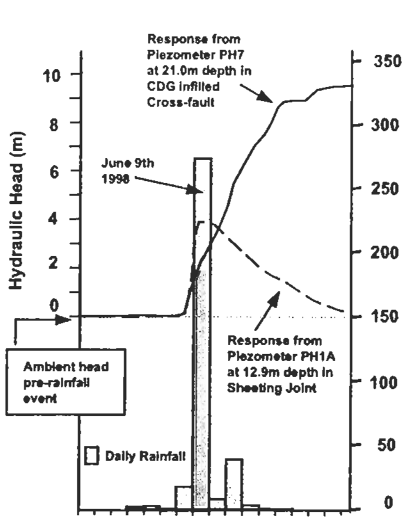


Figure 2: VW piezometer response to a Typhoon event.

In order to gather this key information, the site investigation boreholes were instrumented with vibrating wire piezometer arrays and hooked up to a series of dataloggers. Figure 2, adjacent shows two quite different responses recorded by the VW piezometers as a result of the 280mm single day Typhoon event of June 9th 1998. As is evident, the tip in borehole PH1A in a relatively freely draining sheeting joint shows an almost 3.5m jump in pressure head, with about a 5 day decay period, similar to the responses recorded by Richards and Cowland (1986). The tip in borehole PH7, however shows a steady increase in pressure head with no short term decay behaviour. This head increase to a plateau response maximum corresponds to overflow of compartmentalised mounded groundwater dammed behind the low permeability Fault Zone that cross-cuts through the central section of the TL/S1 slope cut (ref. fault zone and hole locations on Figure 3).

These plots highlight the marked difference in groundwater behaviour from one place to another within a fractured rock mass and emphasise the need for careful examination of such issues in any site investigation in complex rock conditions such as characterise not only the Tuen Mun Highway area but also many other parts of Hong Kong. For the TL/S1, 2 and 3 slope segments where sheeting joint reinforcement was required, careful consideration was therefore given to ensuring that, in addition to providing surplus dowel support capacity to provide an adequate safety factor under the extreme case conditions, sufficient raking drains were incorporated into the designs to mitigate any potential pressure build-up.

Support for the rock mass in the TL/S1 rock cut sections was designed therefore under two guidelines – short term, using temporary construction/excavation pre-support raked tensioned anchors designed to an operational factor of safety of 1.2 consistent with standard Hong Kong Practise (GEO, 1984), and – long-term "permanent", using passive dowels to anchor definable blocks with overall cut slope support designed to a factor of safety of 1.6 based on risk assessment evaluation requirements, necessitating some individual blocks to be checked to factors of safety of 1.8, consistent with appropriate risk based stability assessments. These risk-based factors of safety are higher than the standard values recommended by GEO (1984), Tables 5.1 and 5.4, because of the large number of potential blocks and the high exposure of certain parts of the slope, with limited verge width in the long term.

Two types of permanent dowel were specified; dowels installed within 20° of normal to a sheeting joint which were assumed to act in pure shear, and dowels installed at more than 20° from normal to a sheeting joint which were inferred to act in combined tension and shear. Ultimate and working load capacities for design of individual dowels was based on the comprehensive improved recommendations for dowel capacity and shear deformation provided by Spang and Egger (1990) rather than the Bjurström (1974) approach often previously adopted in Hong Kong.

As excavation of the TL/S1 cuts was to be undertaken immediately adjacent to the active carriageways, in addition to the rockfall protection measures discussed below, a system of pre-support tie-back anchor doweling was installed. Monitoring of the piezometers was automated and an effective warning system was developed through understanding of the influence of transient pore pressure responses on kinematically controlled slope instability. Rainfall events, and subsequent transient increases in porewater pressures were quantified in terms of increased risk of failure, and an emergency response plan implemented.

TL/S1 – FAULT ZONE RETAINING WALL

The retaining wall to stabilise the major fault at TL/S1 represented one of the most significant geotechnical and construction challenges of the project. The wall structure, as finally designed comprised a 1-2m thick vertical reinforced concrete retaining wall of up to 23 m height, backed by a fence of bored, cast-in-place caissons (soldier piles) anchored back to the competent rock mass beyond the fault by double corrosion protected, 36mm diameter tiebacks, bonded 3 m into the hangingwall of the fault/shear zone. The design of the wall was carried out to ultimate limit state (ULS) criteria, with earth pressures being estimated using the Trial Wedge method *(ref. Geoguide 1: Guide to Retaining Wall Design, 1993).*

As illustrated in Figure 3 the fault zone strikes obliquely across the box-cut geometry of the carriageways at TL/S1. Rock, soil and groundwater conditions associated with this fault zone were known from the 1976 original construction to be adverse (Slinn and Grieg, 1979), to the extent that a major failure of this entire zone had occurred when the initial box cut excavation had only been part completed. Data from the comprehensive SI drilling of angled holes and overlapping double holes through parts of the fault structure had confirmed these extremely poor ground conditions. In consequence, excavation procedures for the new cut through the fault zone were specified to be completed by a multi-staged top-down benching and cast-in-place retaining wall construction sequence.

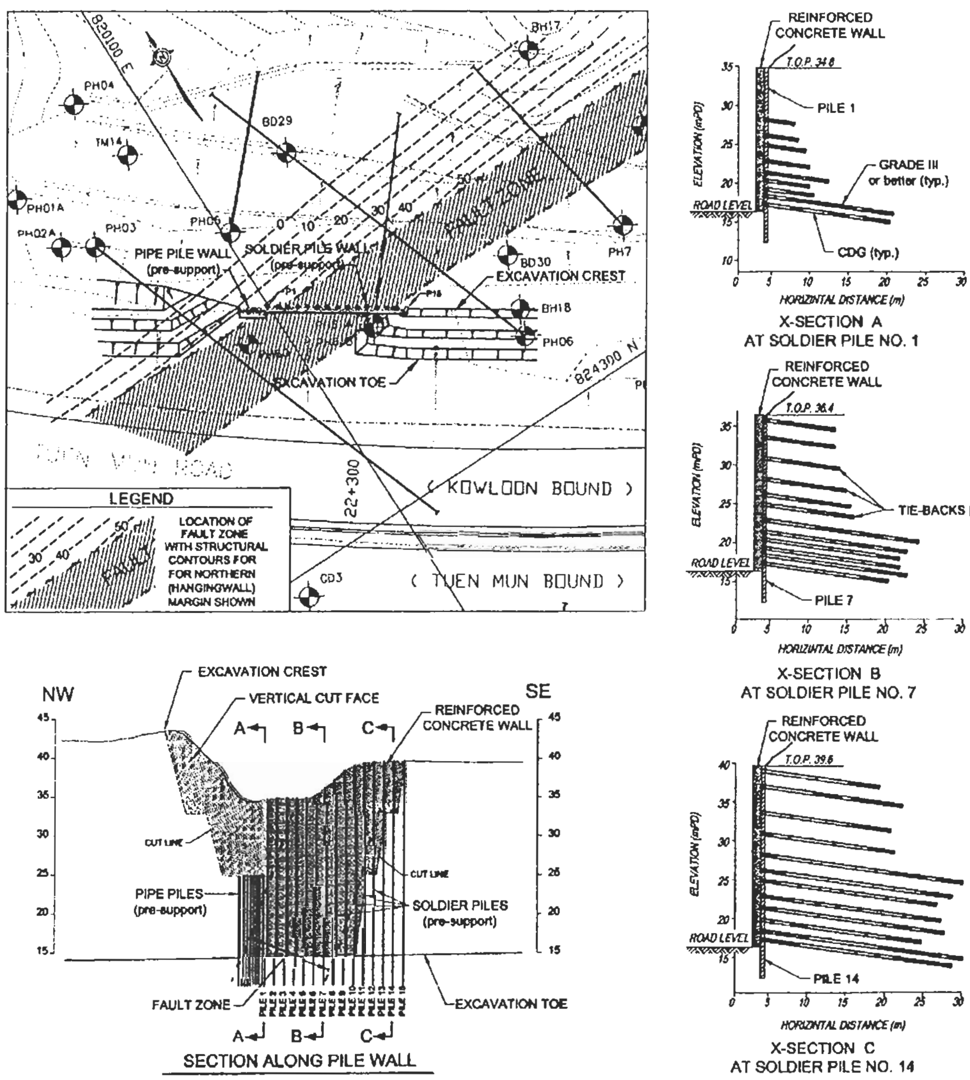


Figure 3: Schematic views of the tied-back retaining wall at TL/S1.

Ensuring adequate temporary pre-support in advance of excavation and retaining wall construction was key, due to the low strength and significant thickness of the fault zone. As shown on Figure 3, the temporary support was achieved using the bored pile wall covering virtually the entire interpreted exposure area of the fault zone. Piles were bored from wall crest elevation to 4 m below the excavation toe. Pile diameter was restricted to 457mm due to rig size limitations related to the extremely difficult access to the drilling location 30m up on the existing cut slope. Most of the drilling went smoothly, however some of the challenges that had to be overcome in the pile drilling included – cutting down in a couple of cases through the steel bar tie-back anchors restraining the existing toe buttress, and – dealing with collapsing pile holes due to the introduction of drilling fluids into the already low strength fault zone CDG material. These challenges were overcome through persistence and flexibility in the construction process, although often substantial delays were endured.

As shown in the photographs in Figure 4, at each stage of the top-down construction sequence, a 2m high segment of the bored pile wall was exposed, allowing drilling of the required tie-back anchors prior to in-place casting of the RC wall itself. Each of the tie-backs was drilled at a nominal 10° down angle into sufficient length of competent grade I-II granite to allow adequate anchor bond length (as shown in the three typical sections included within Figure 4). As excavation and wall concreting was completed on a lift-by-lift basis, drilling of 15° upward inclined raking drains was undertaken to attempt to maintain control of any potential groundwater build-up behind the fault zone. In this way, wall stability was maintained and the construction process simplified.

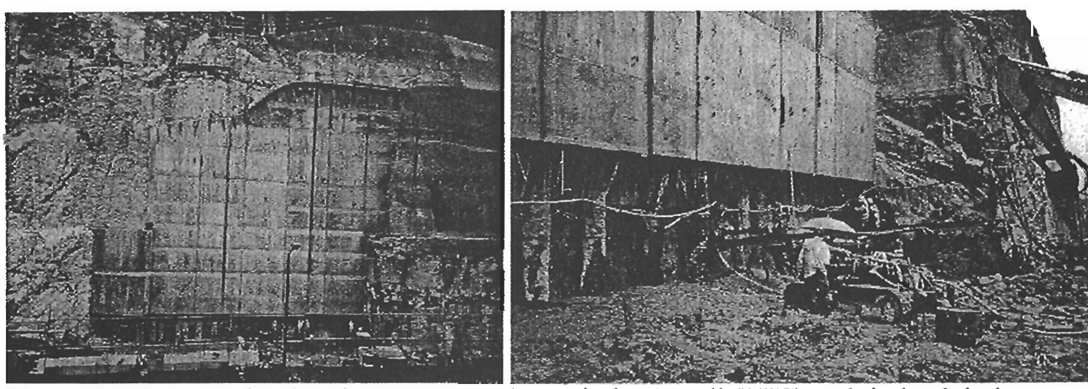


Figure 4: Photographs showing near completed tie-back wall (LHS) and tie-back hole drilling (RHS).

Instrumentation and monitoring

In order to satisfy the strict safety guidelines developed through the comprehensive Risk Assessment Plan for the project, an array of geotechnical instrumentation was incorporated into the design of the retaining wall, aimed at closely monitoring wall performance both during and after construction. In addition to the SI piezometer holes that would remain in operation through at least part of the initial construction phase, the additional specific instrumentation that was placed into the wall zone included:

- Inclinometers installed into 4 of the bored piles;
- Tilt meters and survey nails installed along the wall crest;
- Survey targets installed across the face of the wall;
- Strain gauges attached to selected tiebacks;
- Load cells fitted to the heads of selected tiebacks; and
- A casagrande type piezometer installed within the fault/shear zone immediately behind the wall.

Regular monitoring of this installed instrumentation was carried out throughout construction and into the maintenance period. The majority of the monitoring results to-date indicate that wall performance has been good, with negligible movements. Groundwater behaviour has also followed very closely with anticipated behaviour, in that it was expected that porewater levels within and behind the wall zone might become temporarily elevated by the injection of drilling water during sinking of the piles. Infact, during the period from June to early September 2000 this elevation rise actually happened, but then *in situ* groundwater pressure heads progressively dropped off as construction proceeded. These results are strong evidence of the success of the temporary and permanent internal drainage measures, including temporary raking drains installed in advance of the excavation, and the permanent relief drains incorporated into the retaining wall design.

ROCKFALL & BOULDERS PROTECTION FENCES

Although most of the very hazardous sections of the high rock slopes along the Tai Lam segment of the Tuen Mun Highway had been covered with draped mesh either as part of the original 1970's era construction works, or had been draped in subsequent remedial works campaigns, (mostly associated with clean-up after the fatality and Mediation proceedings), little attention had been previously paid to possible large scale rockfall issues. In view of the long-term hazards posed by possible dislodgement of large, partially stable sheeting joint slabs and/or relict core-stone boulders existing dotted about the natural slopes above the high cuts, the risk-based design approach to the stabilisation programme demanded much higher levels of rockfall protection than had hitherto been practised in Hong Kong. This also applied to the short-term hazards posed by construction of the works themsleves.

The core-stone boulders above the cuts posed a particular problem. Although some of the larger, more easily accessed boulders could be stabilised by construction of site specific concrete butresses and tie-backs, because of the large number of relatively small boulders, it was deemed necessary for both the short-term construction period as well as for long-term (permanent) safety that high capacity rockfall fencing would be necessary to protect the highway from possible loose boulders and other small rockfalls. For the construction period also, the need was recognised that, even though the cut slopes below the proposed new high capacity rockfall fences were going to be stabilised (bolted and meshed during the works), any rockfalls or equipment falling from the cut slopes would have to be prevented from reaching the highway.

Due to the large number of boulders present on the slopes and in particular to assist in laying out the rockfall fences, the natural slopes above and between the slope cuts were initially classified in terms of the overall risk of boulders reaching the carriageways based on the geometrical criteria listed in Table 4. Moderate and high risk areas were further then assessed to determine the general frequency, size and stability of boulders in those areas. Boulders falling into the highest risk category (typically >2m in diameter) that had not already been buttressed were specified to receive additional stabilisation using passive support methods such as dowels, concrete buttresses, and in some cases chunum aprons (to mitigate basal erosion of embedded boulders). This left boulders less than 1m in diameter to be contained by the rockfall fences.

Table 4: Summary of boulder fall path risk and remediation classification systems.

Geometrical/ Fall Path Risk	**Description**	**Boulder 'Remediation' Classification**		
Low Risk	Slopes or valleys dip away from the road or provide adequate catchment area	NA		
Moderate Risk	Long boulder travel distances across vegetated slopes, natural obstructions or shallow dipping ridges or benches	Stable boulders – *no remediation required*	Potentially unstable boulders which could be contained by heavy rockfall catch fences – *no remediation required*	Potentially unstable boulders, too large to be contained by the rockfall fences – *therefore require specific stabilisation treatment in situ*
High Risk	Steep slopes, short travel distances or the lack of obstructions			

Assessments of the potential trajectory paths of probable boulder and or rockfall releases from high on the existing slopes, (undertaken as part of the Mediation proceedings and as part of Golder's initial design brief for developing feasible rock engineering solutions for the identified hazards) had highlighted the need for much higher capacity fences than had previously been used in Hong Kong. Discussions were therefore held with GEO and other controlling Government agencies to allow use of such measures, based on demonstrated precedent of utilising high energy dissipating rockfall fences to mitigate rockfalls and avalanches in Europe and North America. As shown on Figure 5, which illustrates the permanent fence design actually employed along the TMR slopes, these high capacity fences are radically different from the conventional rock fence arrangements included in CED drawings C 2501A and C 2502B, (ref. CED 1994). They typically comprise cable ring netting (originally utilised for anti-submarine nets) strung between posts anchored to the slope with energy absorbing 'break-ring' cables.

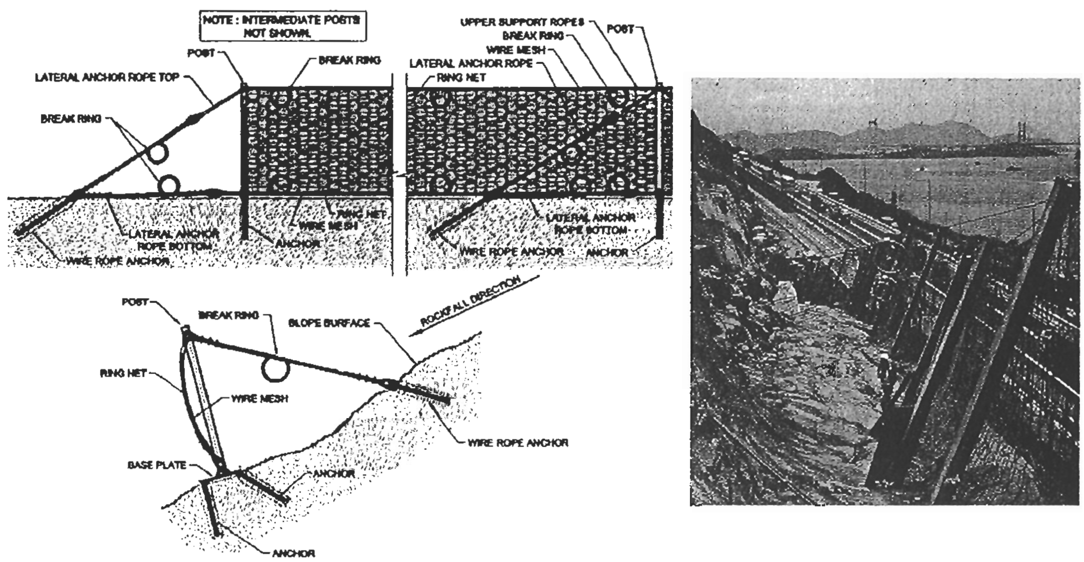


Figure 5: Illustrative example of permanent rockfall fence arrangement as used on the Tai Lam Section, and photograph of actual permanent rockfall fence as installed at TL/S2.

Optimising the positioning and sizing of the fences for both short-term construction control of rockfalls and for long-term boulder and slab slide protection required multiple computer simulation runs of falling, rolling and bouncing boulders. As no commercially available program at the time could adequately undertake the required analyses to the level of design documentation required for GEO submissions, Golder developed an in-house program (Rockfal3) which could address all key parameters required for modelling the most critical bouncing boulder trajectories, namely: boulder diameter, shape, density, release velocity and slope profile, type and roughness and coefficients of restitution for impacts between boulders and the substrate (Golder, 1997). As shown in the typical output plot included within Figure 6, this probabilistic program predicts not only the boulder trajectories, but statistically quantifies boulder bounce height and total kinetic energy along the trajectory paths and thus provides all the output parameters critical for adequately sizing and positioning rockfall fences.

Detailed rockfall trajectory modelling was carried out for each of the major slope segments using site-specific survey profiles of the slopes cut at every 25m along the highway alignment. As will be realised from inspection of the rockfall trajectory profile output shown in Figure 6, obtaining accurate survey profiling of slope geometry is critical to getting reliable and believable rockfall impact predictions. Accordingly, most of the initial rock slope profiles were developed from the terrestrial photogrammetrically prepared DEM, created from helicopter supported photography, corrected as necessary with field survey profiling, using EDM methods. For some of the most hazardous areas, individual specific profiles were subsequently prepared directly from laser target positioning to an abseiling climber traversing down the slope profile.

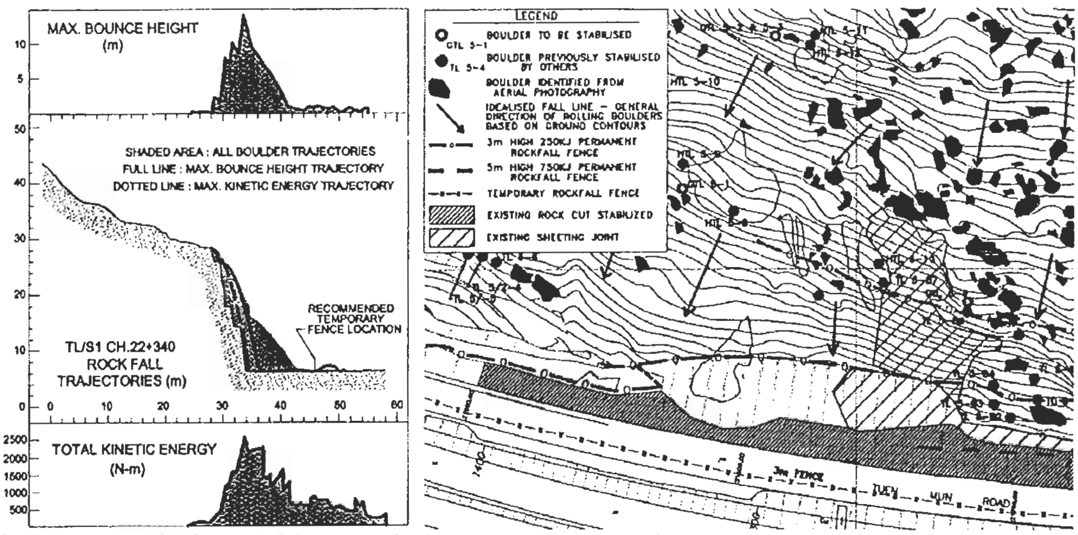


Figure 6: Typical Rockfal3 graphical output for rockfall simulation, and typical example of layout arrangements for fences on a portion of TL/S3.

Trajectory modelling was carried out for two basic conditions –(i) for prevention of accidental construction-related rockfalls, such as had led to the 1995 fatality and –(ii) for long term prevention of degrading slabs or boulders impacting the carriageways. For the construction condition a critical aspect for layout of any temporary roadside fences was the limited available space (margin) between the fence location and the slope toe. Based on the trajectory modelling it was determined that minimum temporary fence height and capacity should be typically 3m and 250kJ, except at the east end of TL/S4 where fence heights had to be increased to 5m to prevent overtopping. Although initial trajectory modelling and the various design reports had evaluated the beneficial effects of a "bounce cushion" in front of the fence to help minimise impact magnitudes, because of the limited width zone at the roadside toe, the contract was tendered with slightly higher capacity fencing without this added safety provision. The Contractor however opted to incorporate a beneficial sand/CDG cushion on top of the carriageway inside the roadside fence for additional safety.

For the permanent fencing, the output from the Rockfal3 code was used to plot optimum design layouts and determine appropriate fence capacities for the fences required in the different slope segments. For all slope areas, the basic procedures were: i) determine fence plan positions relative to slope crest and toe, based on predicted trajectory patterns, ii) establish required fence heights needed to capture the 99% probability level of trajectory impacts, and iii) determine required fence capacity (in Joules) for this 99% probability level. Based on these analyses, in order to provide a simple robust constructable solution, slope positioning of the optimal fence locations were re-arranged to allow use of just three standard permanent fence types – a 3m high, 250kJ fence, and two 5m high fences with 250kJ and 750kJ capacities. In some instances it was found however that fall path length needed to be minimised further still, by using upper, intermediate and lower slope fences. This, in turn provided a substantial measure of redundancy in critical locations.

CONCLUSIONS

Tough constructability issues associated with the widening of the Tuen Mun Highway were overcome through detailed geotechnical investigations, and the design of innovative solutions. As with many other of Hong Kong's problematic high slopes this project has unquestionably raised public awareness and increased the local profile of geotechnical and geological engineering. The project has gone from attracting negative newspaper headlines due to potential litigation and accusations of professional negligence arising from the traffic chaos resulting from the 1995 fatality, to a model example of a well executed construction solution for one of the most difficult and challenging rock engineering projects in the Territory. This is attested to by Hong Kong's first ever high energy rockfall protection fence system and the impressive 23 m high tie-back retaining wall at TL/S1.

REFERENCES

Bjurström, S. (1974), *Shear Strength of Hard Rock Joints Reinforced by Grouted Untensioned Bolts*, Proc. 3rd Int. Rock Mech. Cong., Denver, Vol. 2B, pp 1194-1199.

Carter T.G., Mjerzejewski, J., and Kwong, A.K.L. (1998), *Site Investigation for Rock Slope Excavation and Stabilisation adjacent to a major Highway in Hong Kong*, Proc. Int. Conf. On Urban Ground Engineering, Hong Kong, Paper 14, pp. 10.

Geotechnical Engineering Office, (1984), *Geotechnical Manual for Slopes*, Civil Engineering Department Hong Kong, The Gov. of HK, SAR., pp. 300.

Geotechnical Engineering Office, (1998), *Guide to Retaining Wall Design*, 2nd Edn., Civil Engineering Department Hong Kong, The Government of Hong Kong, SAR., pp. 258.

Geotechnical Engineering Office, (1994), *Civil Engineering Dept. Standard Drawings*, 2nd Edn., Civil Engineering Dept. Hong Kong, The Gov. of HK, SAR.

Dershowitz, W., Lee, G., Geier, J., Foxford, T., LaPointe, P., & Thomas, A. (1995) *Fracman – Interactive Discrete Fracture Data Analysis, Geometric Modelling, and Exploration Simulation*, User Documentation Ver. 2.5, Golder Associates, Seattle.

Dershowitz, W., and Carvalho, J. (1996) *Key-block Tunnel Stability Analysis using Realistic Fracture Patterns*, Proc. 2nd N. Amer. Rock Mech. Symp., Montreal pp.1747-1751.

Golder Associates (1997), *Rockfal3 – A Program for Conducting Rockfall Simulations*, Manual for Ver. 1.0, October, 1997.

Golder Associates (1998), *GoldPit – A Program for Evaluating Rock Slope Stability*, Manual for Ver. 1.21D, March, 1998.

Richards L.R. & Cowland J.W. (1986), *Stability Evaluation of Some Urban Rock Slopes in Transient Groundwater Regime.* Rock Engineering & Excavation in an Urban Environment, Hong Kong, February, 1986, pp. 357-363.

Slinn M.A. & Grieg G.L. (1979), *The Design of Some Constructional Aspects of the Tuen Mun Road*, Hong Kong Engineer, Vol. 4, No. 2, pp. 37-50.

Spang, K. & Egger, P. (1990), *Action of Fully Grouted Bolts in Jointed Rock and Factors of Influence*, Rock Mechanics and Rock Engineering, Vol. 23, pp. 210 to 229.

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岩土工程於公路規劃的應用

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GEOTECHNICAL INPUT FOR THE PLANNING OF HIGHWAY PROJECTS

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摘要

在制訂‘公路斜坡手冊’時，我們曾檢討香港及其他國家在主要公路項目上的規劃程序，並特別關注他們如何進行岩土災害評估。有關的檢討，確定了在岩土方面的議題需以分階段的方式考慮和處理。

土力工程處在二零零零年出版的‘公路斜坡手冊’，建議岩土資料及議題應在公路工程的早期規劃階段進行分析及處理，以便釐定可影嚮工程的嚴峻岩土災害的範圍，及以便選出合適路線和工程設計的最佳組合。此外，岩土災害評估應分兩個階段進行，即岩土災害檢討及岩土災害評估。岩土災害檢討旨在識別和評定嚴峻岩土災害的影響，以及可影響公路工程可行性的關鍵岩土議題。岩土災害評估則旨在根據岩土災害檢討的結果，詳細評定岩土災害的影響，以及可影響公路計劃設計的關鍵岩土議題。

這份文件概述了岩土災害檢討的活動，和有關活動的細節。岩土災害檢討及岩土災害評估的職責範本載於‘公路斜坡手冊’的附件內。

GEOTECHNICAL INPUT FOR THE PLANNING OF HIGHWAY PROJECTS

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Abstract: A review of the current planning process for major highway projects in Hong Kong, particularly with respect to geotechnical hazards, and how other countries conduct geotechnical assessments was undertaken as part of a study during the development of the Highway Slope Manual. The review confirms that geotechnical issues are best considered and dealt with in a staged manner.

The Highways Slope Manual recommends that geotechnical input for the planning of highway projects should be introduced at the early stage of a project to avoid areas with significant geotechnical hazards, where this is possible, and to select an appropriate route alignment with optimum design options. It also recommends that evaluation of geotechnical hazards be conducted in two stages - Geotechnical Review (GR) and Geotechnical Assessment (GA). A GR has an objective to identify and assess the impacts of significant geotechnical hazards and critical geotechnical issues that could influence the feasibility of a highway project. A GA would build upon the findings of the GR and its objective is to assess in detail the impacts of the geotechnical hazards and critical geotechnical issues that could influence the design of a highway project.

This paper outlines the activities in the GR and the level of details to which they are carried out. Model briefs for GR and GA are incorporated as appendices of the Highway Slope Manual.

INTRODUCTION

The planning of a new highway is a multi-disciplinary process which involves transport, highway, environmental and geotechnical professionals. The process starts with the transport/highway professionals putting forward possible route corridors through which a number of different route alignments are to be examined. The preferred alignment and design options along a route are selected based on a balanced evaluation of the relative cost-effectiveness of the possible options taking into account various factors such as transportation requirements, availability and resumption of land, land-use requirements, environmental, drainage and traffic impacts, geotechnical constraints, etc.

HIGHWAY PLANNING

In Hong Kong, the selection of a highway route at the planning stage of a highway project is often driven by the mandatory impact assessments such as the Drainage Impact

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Assessment, Environmental Impact Assessment and Traffic Impact Assessment. Geotechnical constraints, such as the potential hazards from landslides and boulder falls, are not always suitably identified and addressed or are sometimes ignored. It is common that a favoured route could have already been selected in the planning stage without proper geotechnical input with an assumption that geotechnical hazards could be dealt with in the design stage.

In recent years, there are incidents that highways have been seriously impacted by geotechnical hazards, in particular landslides and boulder falls. The cost associated with these occurrences can be high. For example, in the closure of Tuen Mun Road due to rockfall in 1995 and the closure of Ching Cheung Road due to landslide in 1997, there was large public outcry with regard to the social disruption and inconvenience caused to the public. Also, much money and time had to be spent in repair, investigation and litigation.

REVIEW OF GEOTECHNICAL ASSESSMENT PRACTICE WORLD-WIDE

A review on how other countries/territories around the world conduct geotechnical assessment has been carried out as part of a study (Atkins, 2001) for the development of the Highway Slope Manual (GEO, 2000). The countries/territories selected included British Columbia of Canada, California of USA, Nepal and UK, all of which key members of the study team had personal experience, and were also considered to have sufficient similarities to Hong Kong (see Table 1). A common observation from the review of the practice of all these countries/territories is that the geotechnical work for a civil engineering project is carried out in a series of stages, each building upon the results of earlier work.

GEOTECHNICAL INPUT

The review of international practice indicates that great benefits can be achieved if a review of geotechnical aspects is carried out at the early stage of the planning process of a highway, by avoiding areas with significant geotechnical hazards, where possible, and by selecting an appropriate route alignment with optimum design options.

It is suggested that geotechnical input for the planning of highway projects should be carried out in two stages: a Geotechnical Review (GR) and if necessary, followed by a Geotechnical Assessment (GA). The GR/GA would identify and assess the impacts of significant geotechnical hazards and critical geotechnical issues that could influence a highway project. They would also provide the relevant information for determining the scope, land-take, cost and time requirements for the investigation, design, construction and maintenance of the geotechnical works and hazard mitigation measures needed.

GEOTECHNICAL REVIEW AND GEOTECHNICAL ASSESSMENT

For highways projects it is necessary to examine the following in the GR:

(i) The general topographical, geological, hydrological and hydrogeological conditions along and near the proposed highway corridor(s)/alignment(s),
(ii) The nature, range, degree and areal extent of the key geotechnical features and hazards presents,
(iii) The costs, programming and geotechnical personnel requirements for the investigation, design and construction of the geotechnical works and hazard

mitigation measures, and

(iv) The scope of any necessary further geotechnical investigation or assessment (e.g. desk study and ground investigation).

Table 1 Some Key Factors in Geotechnical Assessment Practice in 4 Overseas Territories

Factor	Territory			
	Nepal	British Columbia	California	England
Relief	• Mostly mountainous/ very steep & rugged	• Either mountainous or flat	• Highly varied	• Generally low relief
Climate	• Seasonal monsoonal with occasional intense storms	• Temperate	• Temperate to Arid	• Temperate
Economic Factors	• Mod. High population. • Poor • Small road network	• Low population density • Wealthy • Small road network	• Population spread out • Wealthy • Extensive road network	• Mod. High population • Wealthy • Extensive road network
Geotechnical Hazards	• Frequent Landsliding flooding & erosion	• Landslides rarely soil based, usually rock falls or gully based debris flows	• Large deep seated landslides • Debris flows in mountains • Coastal erosion • Seismic hazards	• Landslides usually associated with periglacial effects & coastal erosion
Geological Factors	• Bare rock. • Deep weathering • Tertiary sediments • Colluvium	• Glacial scouring has removed most weathering products • Glacial drift in valleys	• Deep weathering • Tertiary sediments	• Extensive glacial drift • Tertiary sediments
Notes on Highway Planning	• Low regulatory control • Generally up to consultants	• Regulations under development for "Highway Design Manual"	• Highly Regulated • Route Selection subject to Public Enquiry • Environmental factors are key	• Regulated • Route selection subject to Public Enquiry
Geotechnical Contribution to Highway Planning	• Can be very significant • May dictate route selection	• Geotechnical Assessments not systematically coded	• Geotechnical Assessments not systematically coded, scope determined by project manager	• Geotechnical contribution formally phased

In the GR, geotechnical hazards and related issues are considered in a general manner. Where significant geotechnical hazards are identified, in particular natural terrain hazards, the GR should include a careful assessment to determine whether an adjustment of a route corridor/alignment and/or provision of appropriate mitigation measures are warranted. If alternative route corridors/alignments are to be considered, the assessment should include, where possible, a ranking of alternatives taking into account the possible impacts of the geotechnical hazards and the likely costs of mitigation measures required. The geotechnical ranking, with appropriate weighting, could form the basis for a balanced evaluation of the geotechnical factors with other factors such as land-take, environmental, drainage and traffic impacts, etc., in an overall assessment of alternative route corridors/alignments.

The geotechnical activities in a GA are similar to those of a GR except that these are carried out in greater detail by taking into account additional information obtained from site investigation. Model briefs for carrying out GR and GA are given in Appendix B and C of the Highway Slope Manual (GEO, 2000).

IDENTIFICATION OF GEOTECHNICAL HAZARDS

A key part of the GR is the identification of geotechnical hazards. The most important geotechnical hazards that can affect highways in Hong Kong are slope hazards. These can arise from either man-made slopes (landslides from cut slopes, rockfall from rock cuts, retaining wall collapse and fill slope failures) or natural terrain (natural landslides, boulder falls and rock falls). Examples of the geotechnical hazards that need to be considered are given in Table 2 and Figure 1.

Table 2 Geotechnical Hazards That Need to be Considered for a Highway Project

Existing man-made slope features that could affect or be affected by the project	All substandard features the failure of which could affect the route or could be caused by the works. All features which are up to standard but could be affected by the works (e.g. due to installation of buried water-carrying services, changes in surface and subsurface water flow regime and lateral loads that will be imposed by new highway structures).
Man-made slope features that will be formed along the route	All cut slopes, fill slopes and retaining walls, with particular attention drawn to those to be formed in difficult ground, e.g. steep hillside, adverse geological conditions such as presence of dykes, faults and other weak zones, and adverse surface water or groundwater conditions.
Natural terrain landslide hazards	All natural terrain landslide hazards, including debris avalanches, debris slides, channelised/non-channelised debris flows, boulder/rock falls, gully and other forms of erosion and deep-seated ground movements (see Figure 1), which could affect the route or could be caused by the works. Guidance on natural terrain hazard review and natural terrain hazard assessment is given by Ng et al (2000).
Other geotechnical hazards	All other hazards, including the presence of soft ground and karstic areas.
Hazards associated with the geotechnical works	All hazards associated with construction of the geotechnical works, e.g. rock blasting near sensitive receivers, open or supported excavations including those for cut and cover tunnel construction and temporary diversion of surface water, traffic diversion and/or road closure to mitigate the hazards during construction.

Much information about a site can be obtained from existing records and a desk study of the existing information available is the most cost effective approach to identify those geotechnical hazards that may affect a highway route. Guidelines for carrying out a desk study for site investigation are given in Geoguide 2 "Guide to Site Investigation" (GCO, 1987) and an update on the sources of information for the planning of site investigation is given in GEO Technical Guidance Note No. 5 (GEO, 2001). A site reconnaissance/walkover survey would have to be carried out at selected locations along the routes under investigation, to confirm and supplement the information collected during the desk study. Such reconnaissance work is very valuable. It takes relatively little time to carry out, and it allows a rapid evaluation of the scale of the hazards for comparison of various route options.

EVALUATION OF GEOTECHNICAL HAZARDS

After the identification of geotechnical hazards that may affect a highway route/alignment, the hazards may be evaluated in terms of their potential severity to provide an overall assessment for the ranking of alternative corridors/alignments from a geotechnical perspective. The evaluation of geotechnical hazards in the GR stage would generally be qualitative or semi-quantitative based mainly on existing information. The key geotechnical hazards to be evaluated may include:

(i) The potential instability of new man-made slopes, predominantly large cut slopes;
(ii) The potential for open hillslope landslides of natural terrain;
(iii) The potential for channelised debris flow;
(iv) The potential for deep seated failure of the natural terrain; and
(v) The potential for rock/boulder falls.

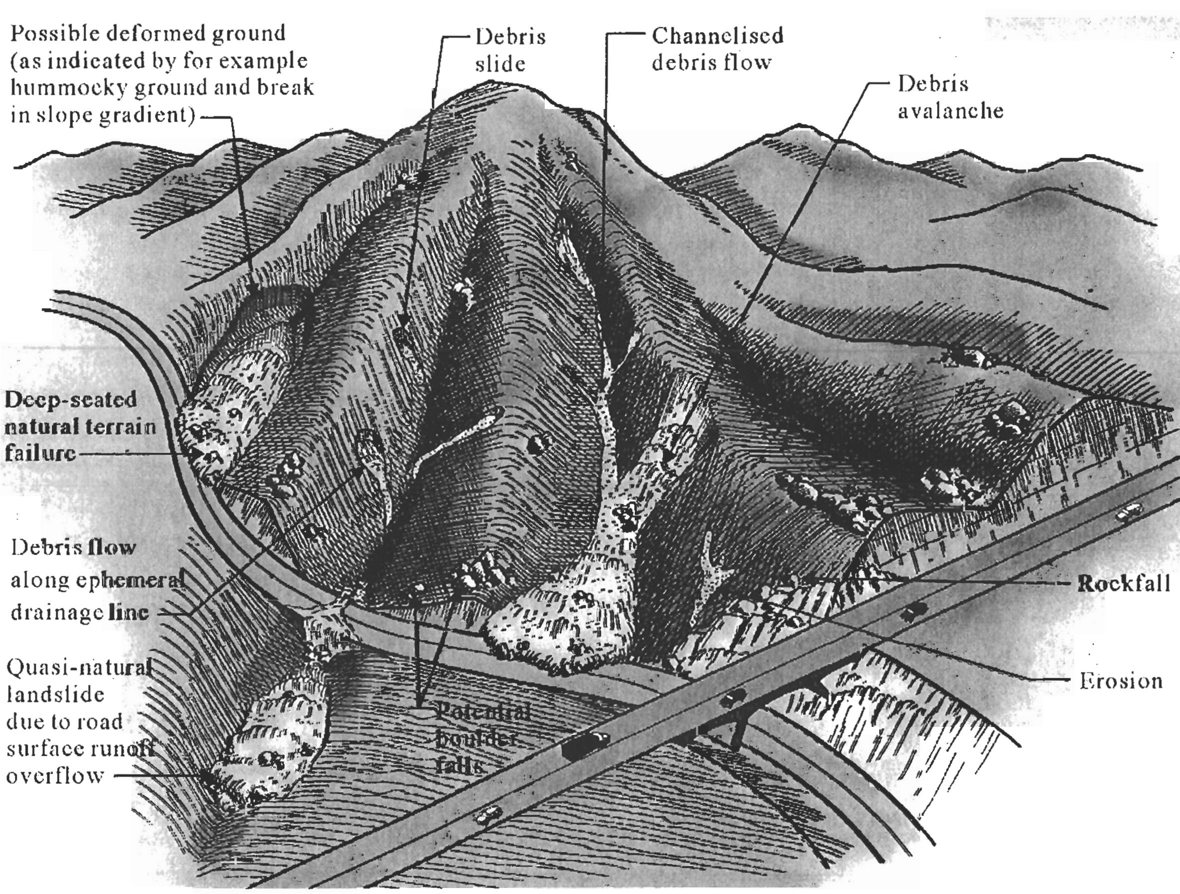

Figure 1 Natural Terrain Landslide Hazards Posed to a Highway

A geotechnical hazard may influence the choice of a route and/or the design of a highway. A particular route may have significant geotechnical hazards which are assessed to warrant consideration of an alternative route. Alternatively, it may be considered worthwhile to mitigate geotechnical hazards through the adoption of an alternative design. The geotechnical engineer (who carries out the GR and therefore has a good understanding of the scale of the identified geotechnical hazards) would be able to guide the project team in deriving an assessment scheme (e.g. a multi-factor analysis) relevant to the project in hand. He/she can also guide the project team in developing the appropriate weighting that ought to be applied to the identified geotechnical hazards in the overall assessment of alternative route corridors/alignments.

DELIVERABLES OF A GR

One of the major output from the GR is the production of a geotechnical feature and hazard map, preferably at a scale of 1:1 000 or 1:5 000, showing the corridor(s)/alignment(s) to be considered. The map should cover all catchments and indicate the source and areal extent of all geotechnical hazards identified, the locations and details of all existing slope features including those to be upgraded and new slope features to be formed under the project, and the principal drainage courses. The map would be useful for the development of a geotechnical hazard mitigation measure strategy and also enable other constraints (e.g. land, existing development and utilities) to be superimposed on it for determining a suitable corridor or route alignment.

As alignments of a proposed new highway are generally not fixed at the GR stage, a rough estimate of the need for man-made slope formation can be made based on the general topography for the purpose of evaluating alternative corridor(s)/alignment(s). The type (e.g. rock/soil, cut/fill), location and the approximate height of the slope features may be estimated for the purpose of preliminary cost estimate and for preliminary consideration of the need for alternative design options (e.g. retaining wall, elevated structure and tunnel).

The GR needs to include a review of natural terrain hazards and to determine if a natural terrain hazard study is required or not at the subsequent stage of the project. The assessment of the impact of the potential hazards from natural terrain is generally qualitative at the GR stage and is very much based on engineering judgement and experience. Interim guidelines for Natural Terrain Hazard Study (NTHS) are given in GEO Special Project Report SPR 5/2000 (Ng et al, 2000). Where a NTHS is considered necessary, the GR should develop a brief for such assessment with the objectives stated and the likely scope, duration and cost provided.

The GR would also determine the objectives, extent, scope, estimated costs and programme of any further geotechnical studies and investigations which need to be carried out at subsequent stages of the project. The GR should establish the types of studies and investigations required to be undertaken during the GA and, where required, develop the Brief for the GA and a GI plan. Sufficient time and funding should be allowed for the planned GA and GI works, which should be adequately allowed for in the overall project programme and cost estimates.

OTHER ITEMS OF WORKS AND ISSUES TO BE CONSIDERED

For highway projects with a high geotechnical content, such as those involving significant slope works, there are usually uncertainties with regard to the geology and groundwater conditions which can have a significant influence on the scope and nature of the works to be carried out. These uncertainties and their implications should be raised in the GR report.

When blasting is anticipated to be carried out near an existing road, the aspects of construction control and safety of road users should be addressed in the GR. Other sensitive receivers should also be identified. In particular, the need for a blasting assessment and/or other additional requirements (e.g. risk assessment) in the subsequent stages of the project should be highlighted.

SUMMARY AND CONCLUSIONS

The GR is a critical activity in the planning stage of a highway project. The GR identifies and assesses the impacts of significant geotechnical hazards and critical geotechnical issues that could influence a highway project. It also provides the relevant information for determining the scope, land-take, cost and time requirements for the investigation, design, construction and maintenance of the geotechnical works and hazard mitigation measures. By carrying out a GR, great benefits can be achieved as geotechnical constraints can be adequately reviewed and addressed at the early stage of a highway project.

This paper has also outlined the major activities in the GR viz. identification and evaluation of geotechnical hazards, and the level of details to which they are carried out.

REFERENCES

Atkins China Ltd (2001). *Improved Methodology for the Planning of Major Highways (Volume 1: Main Study Report). Agreement No. GEO 1/99*, Geotechnical Engineering Office, Hong Kong, 65 p.

Geotechnical Control Office (1987). *Guide to Site Investigation (Geoguide 2).* Geotechnical Control Office, Hong Kong, 362 p.

Geotechnical Engineering Office (2000). *Highway Slope Manual.* Geotechnical Engineering Office, Hong Kong, 114 p.

Geotechnical Engineering Office (2001). *Geoguide 2 - Guide to Site Investigation, Updated Appendix B: Sources of Information, GEO Technical Guidance Note No. 5.* Geotechnical Engineering Office, Hong Kong, 24 p.

Ng, K.C., King, J.P., Franks, C.A.M. and Shaw, R. (2000). *Natural Terrain Hazard Study: Interim Guidelines (Special Project Report No. SPR 5/2000).* Geotechnical Engineering Office, Hong Kong, 132 p.

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香港赤鱲國際機場
地面移動: 回顧與預測使用地理訊息系統

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Hong Kong International Airport Platform at Chek Lap Kok Ground Movement: Review and Prediction using GIS

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摘要

在興建香港國際機場的同時，一個龐大的資料庫收集有關機場平台的沈降及水壓變化情況。 由機場的啓用開始, 這個資料庫仍在不斷更新。 在一九九九年八月, 現存的岩土監察資料平台進行了一次爲期兩年的檢討。 該檢討包括對未來沈降的預測、分析及使用資料庫內的沈降和水壓記錄進行事後分析參數。該次岩土評估已於二零零一年完成, 而其詳細報告已提交香港機場管理局作參考。

該評估還包括將現存的資料庫轉化爲一個可供地理訊息系統程式閱讀的單一資料庫格式。 該地理訊息系統程式容許使用者對不同地點查詢資料庫有關該地點的沈降或水壓動向; 並能簡易地供使用者在大型圖像下查詢及概覽全地盤。 本文目的在說明地理訊息系統如何簡易地供使用者存取及使用資料庫內的資訊。 資料庫可於新資料收集時同步更新。 本文亦包括一些圖表例子展示有關土地沈降、 水壓動向和事後參數的分析。

HONG KONG INTERNATIONAL AIRPORT PLATFORM AT CHEK LAP KOK GROUND MOVEMENT: REVIEW AND PREDICTION USING GIS

Michael S. Hendy[1], K. Y. Wong[2] and Mario Luk[3]

ABSTRACT: A large database of settlement and piezometer readings was collected during the Airport construction at Hong Kong International Airport and the database continues to be updated since the Airport opened. In August 1999 a 2-yr assessment commenced to review the existing geotechnical monitoring data Platform. The review included making future predictions of settlement, looking at the hyperbolic and logarithmic ground models and using the data to carry out back analysis of parameters to be used in analytical models. The geotechnical assessment was completed at the end of 2001 and the results have been published in several volumes in a report to the Airport Authority.

As part of the review, the existing data was transferred into a single database format, which interfaces directly with the GIS-software. The GIS-software permits the user to interrogate the database at individual locations to examine settlement or piezometer trends and also to examine the bigger picture in an overview of the whole site in a simple easy-to-use way. The aim of this paper is to illustrate how the GIS-software has made access and use of the data more user-friendly. The database can be updated as new data is collected. Charts showing example settlement and piezometer trends are presented as well as charts on the back-analysed design parameters.

INTRODUCTION

Figure 1 is a photograph of Hong Kong International Airport platform at Chek Lap Kok shortly after opening. The southern runway and the airport in general, was opened to traffic on 6 July 1998 with the northern runway becoming operational on 26 May 1999.

Figure 1 – Hong Kong International Airport

Filling of the airport platform area to the west of Chek Lap Kok commenced in 1992 and continued until about mid-1995, with stockpiles still being placed in the Central Works Areas (zoned for later development and located in the middle of the site) until the latter half of 1999.

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From the commencement of the platform construction, behaviour of the insitu soils beneath the platform and the fill materials was monitored by a wide range of geotechnical instruments. Comprehensive documentation of the construction was reported in the Proceedings titled: Site preparation for the new Hong Kong International Airport. Ed. G W Plant et al (1998).

For the continuing operation of the airport, it is essential that the international standards for runway and taxiway surface profiles are maintained and that all of the facilities including aprons, tunnels and buildings remain serviceable. It is therefore necessary for the effective continued maintenance of the airport that the settlement behaviour of the platform continues to be monitored, and that future settlement trends are identified. This in turn allows a systematic maintenance schedule to be developed.

Instrumentation includes piezometers, surface settlement markers and automatic and manual extensometers. Much of the data goes back to the formation of the platform in the mid 1990's and the database is therefore becoming a unique record of the platform performance over time as the data collection has continued into the operational phase of the airport. A Geographical Information System (GIS) model has been developed to handle the data and to allow continuing upgrading of the database. The software adopted for this project is a commercially available program, MapInfo. Other software capable of providing similar processing functions is available in the market place and the choice of software is related to both the cost and complexity required. All the settlement data collected from the Airport Platform has been entered into the GIS from the Excel spreadsheets or ASCII files on which the raw-data is stored. From the Excel spreadsheets, the data is processed into a form where it can be interpreted and predictions made. Information from all instruments, locations and readings can be viewed interactively on a screen. The GIS incorporates the facility to interrogate the readings using either a natural, root or logarithmic time basis or viewed using a hyperbolic relationship.

ZONING PLAN

The original zoning plan was – in part – a management tool developed for the construction phasing. The zoning plan concept was developed to group areas of similar properties, history and ground conditions and has been retained, as much of the historical data is referenced to the zoning plans with the original layout only slightly modified. The current form of the Zoning Plan is illustrated in Figure 2. It is noted that the north and south runways are divided into parts of approximately equal length with separate zones for the Eastern Vehicular tunnel and the original islands of Chek Lap Kok and Lam Chau.

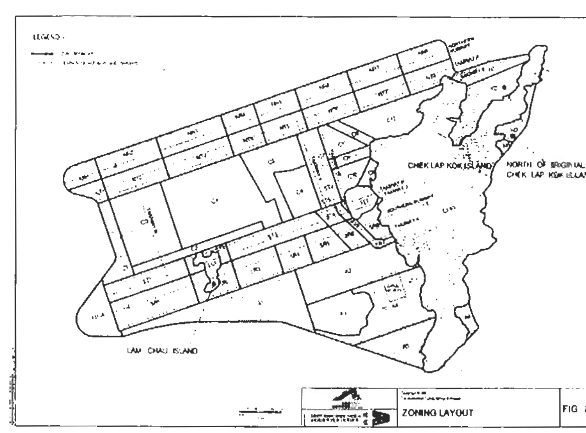

Figure 2 – Zoning Plan for Chek Lap Kok

GEOLOGY AND RECLAMATION FILL

Although the geology of the site has been reported previously, eg Plant et al (1998), a brief description of the platform foundation is given below. A report by the British Geological Survey (BGS, 1994) divided the superficial geology (overlying completely decomposed granite) of the site into three main groups:

- Marine muds of the Hang Hau Formation
- Alluvial clays of the Sham Wat Formation
- Alluvial sands and clays of the Chek Lap Kok Formation

Hang Hau Formation

The Hang Hau Formation (HHF) is described as a fairly uniform deposit of very soft greenish grey silty clays of marine origin; this deposit has been almost entirely removed from below the airport platform by dredging operations prior to the placement of fill. The deposits were completely removed along the line of the seawalls. The dredging operation may have resulted in up to two metres of the formation being left in place elsewhere on the seabed, but some displacement and inter-mixing during filling is expected to have occurred.

Sham Wat Formation

The Sham Wat Formation (SWF) was revealed to the west of Lam Chau island, between Lam Chau and Chek Lap Kok and in the north-eastern corner of the platform area. The SWF has been subdivided into three units and although it may generally be described as a firm silty clay it does contain softer horizons. The unit lying between the two former islands contains sand beds up to 1m thick.

Chek Lap Kok Formation

The Chek Lap Kok Formation (CLKF), which forms the majority of the superficial deposits remaining in place after dredging, has been sub-divided into three main units and several sub-units. These sub-units are amply described in the reference ed. Plant (1998) and were developed by the BGS (1994). Essentially the upper unit generally comprises a firm to stiff clay but is softer in places with some sandy channels. This unit is described as a "crust" as a result of it forming an old land surface. The middle unit comprises predominantly horizontally bedded soft to firm slightly silty clays. Locally there is evidence of a further "crust" beneath the base of the unit. The lower of the three units is the most heterogeneous and underlies most of the platform and lies directly upon rock. It has been divided into four sub-units, but essentially comprises firm to stiff silty clay sub-divided with medium dense to dense fine to coarse-grained sand/gravel towards the base.

Types of Fill Materials

Seven categories of fill material were used during construction of the platform, although the bulk of the material placed comprised one of the following four. Figure 3 is a summary of the Fill Allocation Plan.

Type A - As-blasted rock with a maximum size of 2m and a fines limit of 5%

Type B - Excavated soil or rock with a maximum size of 300mm

Type C - Marine sand with a fines content of 20% or less

Type A/B - Types A and B fill is predominantly Type A and highly variable.

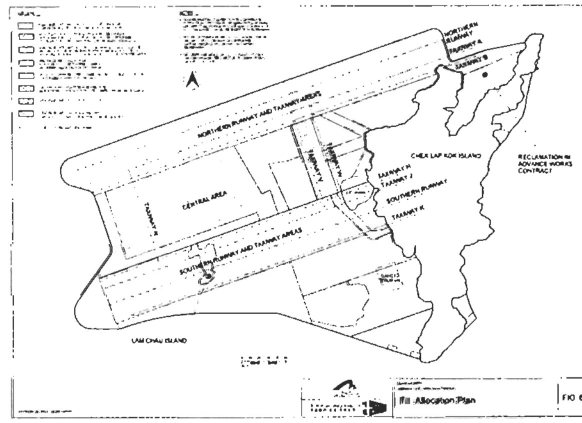

Figure 3 – Fill Allocation for Chek Lap Kok

It is noted that boulders of up to 2 m were permitted within the Type B fill at the Engineer's discretion. Additionally, although up to 20% fines were permitted within the Type C fill material, in practice the actual percentage of fines was much less. Fill was placed by a combination of bottom dumping, rainbowing or end-tipping. The platform was capped mainly using 2m of Type C material except in many eastern areas where Type B was used.

SETTLEMENT REVIEW AND PREDICTIONS

The on-going collection of data is carried out systematically across the site with an emphasis on the two runways and taxiways for operational reasons. A total of more than 1300 datapoints currently exist and continue to be monitored. The extent of the data available and hence the difficulty of trying to track, monitor and access the data manually is a significant resource problem. The knowledge of the ground conditions and fill types together with the individual settlement and piezometer records can be used to carry out back-analysis of soil parameters at individual locations and hence make predictions of residual settlement at particular time frames.

The purpose of the project has been to access datapoints easily and to use the prediction tools to determine residual settlement and settlement trends. By having the data stored on GIS, predictions are easily tabulated by a chosen method and thus updates of predictions can be made with minimal work. The results of the monitoring data are available within the GIS program and can be plotted as summary graphs with the typical ground conditions as presented on Figure 4.

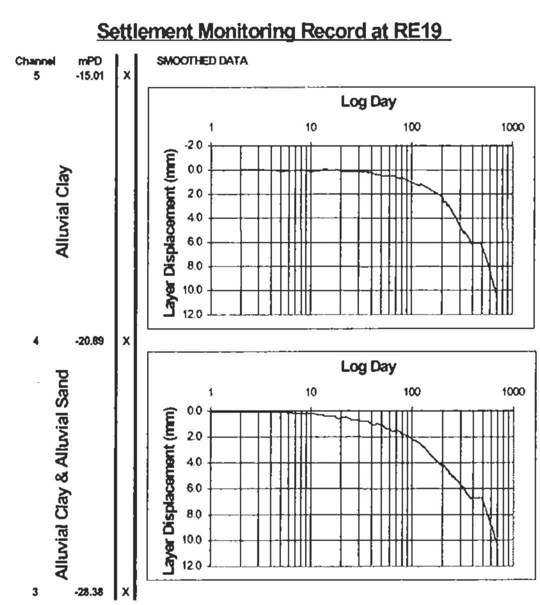

Figure 4 – Settlement vs Log Time

GIS PROGRAM

The GIS software used for this project is MapInfo Version 5.5. All programs are saved as MapBasic executable. Although detailed knowledge of MapBasic is not necessary to run the program, a working use of the GIS software is necessary for effective operation. Additional software has been developed for the project so that plans and graphs can be easily opened onto the computer screen and individual data points can be interrogated. A GIS User Manual has been provided so that the program can be readily used by three different categories of personnel: operational users; system administrator and engineering user. These categories each have access at a different level into the program software. Use of the program has been made as simple as possible. For example, layouts of the site are saved as Workspace files. Thus to view the Zoning Layout, choose the zoning-layout.wok file in the Open Workspace dialogue box and the Zoning Plan will open on the screen.

Surface Markers and Extensometers

The interpretation of creep parameters for predicting future settlement depends on curve fitting and it is frequently the case that insufficient data is available because predictions are being made during the construction process, for example to determine the removal of surcharge as part of the ground improvement. Curve fitting in these typical construction cases is therefore undertaken with only a few months' of data. Historically the use of a logarithmic model has been most widely adopted for the prediction of long-term residual settlement (Hendy et al (2001)). The unique set of data for this site spans a period of several years after removal of surcharge. There is therefore sufficient data to provide greater confidence in the results. After examining different methods, the data plots indicate that the best-fit line follows a log-time curve. Curve fitting using computer generated 'best-fit' lines was considered for the assessment of creep parameters. However it was also considered critical that back-analysis permitted the engineering user to examine all of the factors at each location – ground conditions, construction history etc – and a manual process has therefore been adopted to establish the back-calculated creep parameters.

The maintenance of the runway depends on providing accurate predictions of settlement over short and long-term periods (within the limitations of current analytical methods) and the manual procedure allows the engineering assessment to accommodate factors that would not be readily addressed in a fully automated system. Figure 5 is the basic algorithm used for developing the software and Figure 6 is a sample of a settlement time prediction at taxiway H. Inspection of Figure 5 illustrates how the interface with GIS is required at the beginning and the end of the procedure, broken by the manual interpretation process.

Inspection of Figure 6 illustrates that, as expected, movement at Lam Chau Island has been minimal with small residual settlement predicted for the coming few years either side of Lam Chau island. After calculating the total and differential settlement from a surface marker location, the system can create a graph of the results for each marker. A plan of the whole site can be displayed on the computer screen and the chosen location selected. The system allows a choice of graphical presentation: for example, settlement against time or log time or a hyperbolic relationship. Once the form of the plot has been determined, the graph of the data can be displayed.

1. Update Survey records in GIS Model

2. Data Processing in GIS
- Production of log-time settlement plots

3. Predict Future Settlement
- Determination of design settlement rate
- Calculation of creep factor
- Estimation of residual/differential settlements

4. Analysis of Results
- Statistics of creep factors in each zone

5. Production of Zoning Plans
- Input creep factors to GIS
- Input settlement predictions to GIS
- Input results of statistical analysis to GIS

6. Prediction of Future Maintenance Requirements
- Compare compliance with seviceabiiity criteria of various facilities
- Recommendations survey/maintenance

Figure 5- Simple Algorithm for Settlement

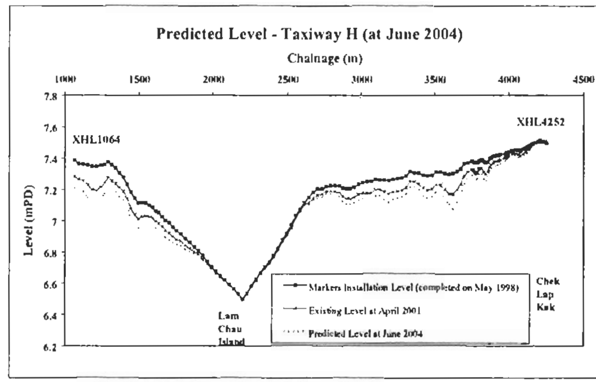

Figure 6 – Sample Settlement Prediction for Taxiway H at June 2004

1. Creep factor is the gradient of the best-fit straight line per log cycle of time.
2. Lam Chau is the location of the original Lam Chau Island.
3. Taxiway H is the airport taxiway parallel to the Southern runway, adjacent to the Central Works Area.

Piezometers

Similarly, the system allows the plotting of graphs for piezometer data. Calculations can be performed using tip pressure or excess pore water pressure and the resulting chart can be displayed (and printed): for example, in Figure 7 the piezometer tip pressure is plotted against the log of elapsed days. Graphs can also be edited in the graph window of the GIS program if different forms of output are required. Figure 7 clearly indicates that the excess pore water pressure has been substantially dissipated for some time.

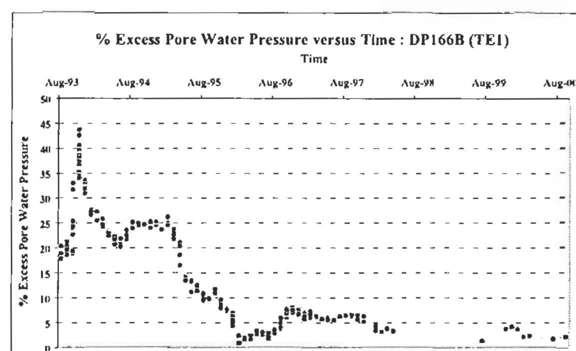

Figure 7 – Sample of Time Excess Pore Pressure Plot

New Data

It is noted that the program always refers back to certain MapInfo files. These MapInfo files can be edited because each time the program is run, it will be necessary to update the files. New data can be added to the system. This is essential in order to use the latest survey data for plotting. As the incoming data is in Excel format files, the MapInfo system has been programmed to interface directly with these files and thus to update the MapInfo tables.

Additional datapoints can also be added at any time: for example, surface markers, piezometers etc.

Once new data has been added to the system, the total and differential settlement files can be updated. The surface marker program can be used to calculate the total and differential settlement. Separate programs are available for calculations with the different instruments. For example, piezometers and extensometers.

Back-analysis

Part of the requirement of the commission was to back-calculate design parameters from the actual field data. Back-calculated, best-fit lines were drawn to the data and hence creep factors calculated at each location.

Once the creep factor has been determined, the analysis is presented graphically. For each of the zones around the site, a plan is marked up with the creep factors thus indicating the variability of the creep for that zone. An analysis of the creep is presented and this is shown as a histogram. A key plan of the site is also shown. An example is presented at the location of the former Lam Chau Island on Figure 8.

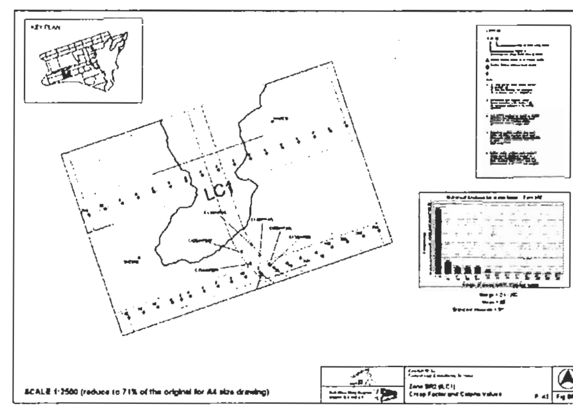

Figure 8 – Plan of Creep, Lam Chau Zone

Figure 8 shows that the creep factor can be calculated every 30m or so along the runway where data is available on the lighting cans. The histograms are plotted to give an immediate visual impression of the variability of the data and therefore the predictions. Inspection of Figure 8 reveals that, as expected, the creep factor is very low at Lam Chau – indicative of minimal movement.

CONCLUSIONS

The Hong Kong International Airport is a world-class project. The extensive geotechnical instrumentation that was installed during the construction period continues to provide valuable information on the long-term settlement performance of the platform. The GIS program that has recently been developed and customised for updating and managing the data provides an invaluable tool for determining the long-term maintenance requirements for the Airport Platform. This is particularly important for the runways and taxiways to enable efficient planning of maintenance. The available data also provides a unique source of long term settlement behaviour of fills of varying types as well as the underlying deposits.

REFERENCES

Hendy, M. S., Ching P. T., and Muir I.C. (2001). "Creep Settlement in Reclamation Sand Fill, A Case Study." Geotechnical Deformations and Movements, HKIE Annual Geotechnical Seminar, Hong Kong.

James J. W. C., et al. (1994). "An interpretation of the marine geology for the replacement airport at Chek Lap Kok, Hong Kong." British Geological Survey (BGS). BGS Technical Report WB/93/34R.

Muir I.C., Hendy M. S and Ching P. T. (2001). "Creep Settlement in Reclamation Sand Fill, Analytical Review and Early Predictions." Geotechnical Deformations and Movements, HKIE Annual Geotechnical Seminar, Hong Kong.

Plant, G. W., Covil C. S., and Hughes R. A. (1998). "Site preparation for the new Hong Kong International Airport".Thomas Telford publ.

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香港國際機場 - 機場島維修與發展上的岩土工程事務

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Hong Kong International Airport – Geotechnical Aspects for Maintaining and Developing the Airport Island

Mario C. W. Luk
Airport Authority Hong Kong

撮要

本文旨在簡報位於赤鱲角機場人工島維修與發展上之岩土工程事務，並重點提出近期之發展。

筆者首先交代背景，並勾劃出各類機場設施維修與發展上岩土工程事務的概要。其中包括(一)因應各類設施特性而作出之差額沉降的監察及預測，(二)斜坡，擋土結構物，海堤，設施基礎，以至一般土地的定期監測，與及(三)維修保養工作之施行。跟着，筆者匯報機場島上之岩土工程儀器監測工作的整體概況。與其類比，可說規模龐大。最後，亦對作爲岩土工程事務基建的資訊系統作出回顧與前瞻。

Hong Kong International Airport – Geotechnical Aspects on Maintaining and Developing the Airport Island

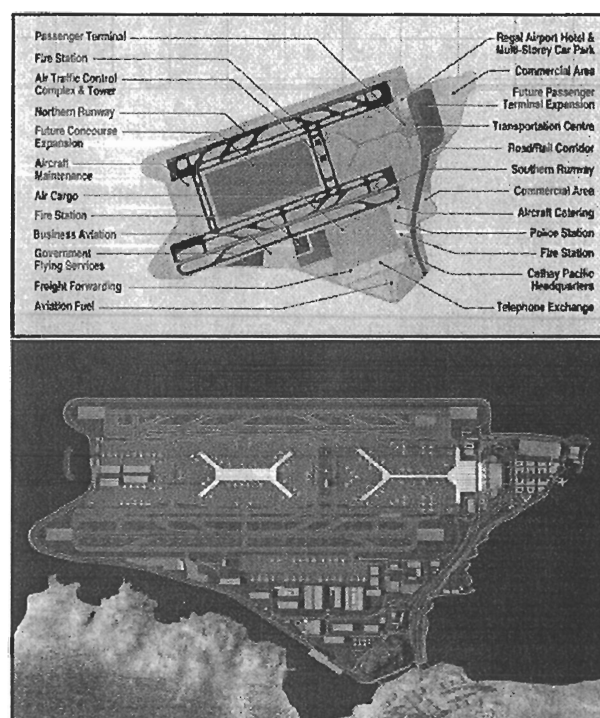
Mario C W Luk [1]

Abstract: This paper aims to briefly report on various geotechnical engineering aspects of maintaining and developing the airport man-made island at Chek Lap Kok, and highlights on some recent developments.

The paper firstly sets the background and outlines geotechnical aspects in maintaining and developing various airport facilities, including (1) differential settlement monitoring and predictions for facilities of various criteria; (2) regular monitoring on slopes, retaining structures, seawalls, facility foundations and general ground; and (3) implementation of maintenance works. And then, reports are given on the geotechnical instrumentation work for the Airport Island, which has been a large-scale work of its kind. Lastly, information system as an infrastructure for geotechnical engineering is also reviewed.

Development of The Hong Kong International Airport

Figure 1- Airport development plan.



The current Hong Kong International Airport (HKIA) island (referred to as the Airport Island hereinafter), 12.48 square kilo-meters in area (about 1% of Hong Kong territory's total area in 1990), was constructed in 1991 to 1995.[2] Construction of major airport buildings and facilities started as early as 1994. The airport was opened on the 6th July 1998 with one runway, one passenger terminal building, two cargo terminals, one aircraft maintenance hanger, and other associated facilities. The second runway and the Northwest Concourse were opened in 1999 and 2000 respectively. The airport will be fully developed in 2020 when the second

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[2] Geotechnical aspects of constructing the airport island has been widely discussed in many occasions, and such are not to be repeated in details in this paper unless required for easy reference in relation to this paper. Background information and construction information of the airport-island was well documented in an Airport Authority publication (G Plant et al, 1998). A brief of various key engineering aspects in construction and development of the airport island can be found in a recent paper by Luk (Luk, 2001).

phase is completed according to Master Plan 2020 of the Hong Kong International Airport. Figure 1a (top) shows the current airport facility zoning plan. Figure 1b (bottom) shows a development plan for the Airport up to the year of 2020.

The reclaimed land can be further divided into seven sub-zones. They are (i) the North Commercial Area for future development, (ii) the Northern Runway Area, (iii) the Mid-field Area consisting the west apron area and land reserve for second concourse, (iv) the Aircraft Maintenance Area, (v) the Southern Runway Area, (vi) the Business Aviation Area, and (vii) the Cargo Area. Each of the zones has its particular underground geology, development and maintenance history, and unique service criteria.

GEOTECHNICAL MAINTENANCE ASPECTS OF THE AIRPORT ISLAND

Since January 1997, maintenance of the reclamation has been the responsibility of the Airport Management Division (AMD) of the Airport Authority Hong Kong (AAHK). The Authority's Geotechnical Engineering Section is dedicated to looking after the all the geotechnical engineering aspects of airport maintenance and settlement issues. In addition to that, planning and development are another essential functions for the Section.

The maintenance works comprise of planned preventative works and reactive remedial works. The Airport Island can also be divided into three areas – reclamation, earth-retaining facilities, and marine areas for maintenance monitoring. Table 1 summarises major tasks of the current Geotechnical Engineering Section in Airport Island maintenance and development.

Table 1- Tasks and Means in Airport Island Geotechnical Maintenance and Development

Objectives		Actions
1	Carry out instrumentation for the airport island	• Maintain an in-house instrumentation team. • Carry out regular instrument monitoring on sub-surface geotechnical instruments and surface level markers. • Carry out instrument calibration and maintenance.
2	Provide geotechnical information for land development, facility planning and maintenance.	• Collect comprehensive geotechnical information, ground investigation records, construction and maintenance records. • Design and implement new ground investigations and instrumentation schemes. • Establish a comprehensive and realistic picture of airport ground behaviour, base on regular reviews and back analysis on instrument monitoring data. • Carry out ground settlement analysis and predictions at locations of important airport facilities. • Produce desk study reports on ground conditions. • Provide other information services.
3	Prevent land and foundation hazards (such as failures of slope, retaining structures, and reclamation perimeter (seawalls), damage of underground soil filter layer, and building apron	• Regular instrument monitoring. • Inspect for maintenance on all slopes and retaining structures after major heavy rainfall or typhoon. • Inspect general ground and buildings regularly to discover signs of possible failures, and carry out maintenance. • Regular seabed level survey and review on seawall stability. • Monitor ground investigation activities and excavation works to prevent damage to underground soil filter layers and geotextile filter layers.

	subsidence).	• Regular engineer review on inspection results.
4	Search for solution for facility failures caused by geotechnical factors.	• Carry out special studies related to geotechnical engineering. Work in collaboration with other engineering disciplines. • Procure services from specialist consultants and contractors.
5	Generate innovative solutions to current problems.	• Carry out research studies on engineering subjects with potential benefit to AAHK.
6	Lower the cost of planning for developments.	• Plan and implement ground investigations in areas for large-scale developments. • Plan and implement instrumentation to obtain long term ground movement information for large-scale developments in reclaimed areas. • Provide geotechnical reports and information service for tendering and design purpose.
7	Develop proposal for new facilities.	• Review and give advise on geotechnical engineering aspects for new proposals for development.
8	Promote cost efficiency	• Establish technology study group to review and invent new technologies for airport environment. • Establish research and development collaboration with external academic bodies and technology research groups. • Establish new systems and procedures to maintain high efficiency and standards.

To maintain efficiency of the above tasks, comprehensive design data, construction records, ground information and instrumentation data are entered into a well-structured geotechnical information system for easy retrieval. This is to be discussed later. The Authority's geotechnical engineering section also owns its instrumentation team to guarantee quality of data collected from the instruments, and to allow efficient maintenance on the instruments so that continuous data are available from all desirable locations. Quick response with high level of efficiency and safety has been maintained for instrumentation / investigation in critical areas with strict time constrains.

As various geotechnical aspects on design and construction stages of the Airport Island have been widely discussed in various occasions, discussions in the following aim to give more details on geotechnical aspects during airport operation stage as updated information, also as a supplement to a few well-known topics about geotechnics at HKIA.

(1) Differential settlement monitoring and predictions

Settlement monitoring and prediction work for the Airport is quite different to that for an ordinary building site, because of unique working environment and service criteria for some of the airport facilities. One of the typical examples is maintenance criteria on runway profile which requires that there is no more than 25 mm level difference from the runway profile over any distance of 45 m along the runway. Otherwise, surface re-profiling is required. Major constraints on developing a monitoring system for the runway are:

(1) 24-hour airport operation – allows only two mid-night windows of 8-hours-work (including stand-by time and clearance) every week for each runway.

(2) Runway pavement surface restrictions – survey nails are not allowed due to aviation safety requirement, and marks on pavement surface are not practical due to heavy aircraft tire marks and maintenance resurfacing. Subsequently, runway navigation lighting cans built in the pavement in 15 m to 30 m centres along runway centre-line are used as survey points.

Meanwhile lighting cans on runways and taxiways are surveyed at least twice a year. Profilometry surveys, measuring relative level at 305-mm-intervals, are also taken once every year for runways and taxiways along their centrelines and offsets. For runway section with 30-m-interval lighting cans, settlement is interpolated between two lighting cans with reference to profilometry survey data, such that assessment of settlement in 45m length is possible. More frequent measurements at close grids will be carried out for potential problem areas identified.

Besides operational purposes, settlement monitoring and prediction is also carried out for future developments. For instance, in the land reserve area for future concourse in the mid-field area, potential settlement is monitored by a special instrumentation scheme. This scheme comprises of over 900 settlement markers, 9 extensometers and 14 piezometers. Nine drill holes to rock level and laboratory tests on soil samples retrieved were carried out. Surface monitoring points in 50-m grid covering the whole site is established to give general settlement parameters on various fill materials overlying different in-situ geology. 25 m spacing is used along future tunnel alignment. Surface monitoring points in 10-m grid and clustering with an extensometer and a standpipe are established in four typical geological areas to predict potential differential settlement to future foundations. This scheme will provide valuable realistic data for future designer in considering for low-cost shallow-foundations to be built on reclaimed land.

(2) Regular monitoring on slopes, retaining structures, seawalls, facility foundation and general ground, and

Besides monitoring differential settlements, other airport facilities are also monitored for safety and regulation compliance reasons. About 50 slopes and retaining structures of height exceeding 3 meters are monitored regularly according to GEO guidelines to prevent undesirable failure. A register and database is established for this purpose. Seawall stability is monitored by surface markers behind their crests, coupled with inclinometers and extensometers at strategic locations to pick up movement at possible failure surface. Hydrographic survey is carried out annually to monitor the seabed condition at seawall toe level. Foundations for other less critical facilities are simply monitored by surface markers at external columns. Instruments are also installed on general ground of different geological conditions to collect ground movement parameters for general reference and parameter studies. Unexpected ground movements are monitored using ad hoc instrumentation schemes.

(3) Instrumentation for the Airport Island and Facilities

Apart from the instruments inherited from project stage, new instruments were installed for operation stage and for planning of future development.

Most instruments installed within or close to facilities footprints were unavoidably damaged in construction stage, except those received particular protection. Surface markers are most vulnerable to construction damage. Subsurface instrument monitoring points are difficult to relocate, unless permanent sensor and cabling can be installed to connect the instrument to another access point, like piezometers.

In the operation stage, instrumentation is implemented for two major purposes. First is for collecting data to monitor important facilities, such as runways and taxiways, against their performance criteria. It also provides important information for special studies on airport facilities. Second is for collecting ground information and establishing ground movement trend for the planning of future facilities on the airport, including major development areas on reclaimed land, such as the mid-field area between the two runways. For instance, the instrumentation scheme for the mid-field area provides information for both runway areas and for future concourse development.

Monitoring and maintenance of all the existing instruments as listed in Table 2 are currently carried out by the Authority's Geotechnical Engineering Section (comprises of an engineer, a superintendent and a technical officer), except surface markers are monitored by Survey Section. Installation of surface mount instruments and instruments buried in shallow depth is carried out by the Geotechnical Engineering Section with assistance from the civil maintenance contractor.

Monitoring intervals of various instruments vary according to magnitude and period of change of measuring parameter. For example, settlement for general reclamation is monitored twice a year, while soil pore pressure subject to tidal effect is measured every 15 to 30 minutes.

All the data collected are regularly reviewed by engineers for particular purposes, such as instrument performance, scheme review, and defects monitoring, etc. and reviewed annually by consultant to give prediction for airport facility maintenance purpose. All these reviews require comprehensive knowledge on ground conditions, construction and maintenance history, facility's service limits, instrument performance and familiar on the data conversion processes. Careless omission or mistake in any part of them can lead to an entirely different conclusion, particularly in the Airport's complex instrumentation system.

Table 2- Existing geotechnical instruments on the Airport.

Monitoring Subject	Instrument		Logging Method	
	Type	Existing No.	Manual	Auto
Ground conditions during construction	Surface settlement markers, inclinometers, extensometers and piezometers	<10% of total number	Majority	Short time only.
Underground structures	Crack width gauges	21	21	0
	Joint width gauges, linear displacement transducers	47	46	1
	Tilt gauges, tiltmeters	8	4	4
Lateral ground movement profile	Inclinometers	70	70	0
Soil settlement profile	Extensometers (magnetic-ring)	37	33	4
	Extensometers (rod)	10	0	10
	Extensometers (steel-ring)	36	36	0
Soil pore air pressure	Pressure Transducers (vibrating-wire)	17	0	17
Soil pore water pressure	Piezometers (pneumatic)	14	14	0
	Pressure Transducers (vibrating-wire)	200	30	170
Ground water table	Standpipes	36	36	0
Tide level	Tide gauges	2	0	2
Atmospheric air pressure	Barometer	1	0	1
Surface Level Markers		3094	3094	0
Tunnels	Bolts	*161*		
Runways	Lighting cans	*372*		
Taxiways	Lighting cans	*1372*		
Buildings	Bolts	*124*		
Seawalls	Bolts	*88*		
Developments	Nails and rods	*640*		
General	Rods, nails and bolts	*367*		

It is worth noting that, by experience from the instruments installed on the airport, adoption of automatic logging systems for instruments has to be carried out with extra cautions. Not only because of the relatively high capital cost, but also because of high maintenance costs under stringent working condition similar to that in the Airport. As all the instrument inspection pits in the airfield have to be built to below ground level, all automation systems are under prolonged immersion in water in rainy days. In addition, when instrument automation systems have to rely on battery power supply, batteries discharge unexpectedly quickly in hot and humid weather and have to be replaced every month. The cost, effort and high frequency in maintaining the auto-logging systems may fail the original intention of installing them.

Different from construction stage when instrument maintenance is carried out by contractors, instrument routine maintenance works is currently undertaken by the Authority's geotechnical engineering section. Decision was made not to out source the work because of the large number of instruments, harsh working conditions, high standards in safety, quality and environmental requirements but relatively low budget. Same as most of the other geotechnical maintenance works, assistance is obtained from the Authority's civil works term contractor for instrument maintenance works. The contractor can be mobilised in relatively short notice, and is ready to work at any time in the 24 hours of everyday. Specialist contractors are available as sub-contractors under the civil work contractor.

INFORMATION SYSTEM FOR ENGINEERING

To handle large amount of ever growing data from geotechnical instruments, in addition to ground information and construction records, information system has been an essential part for geotechnical engineering on the Airport Island. A good information system is the foundation of an efficient geotechnical engineering team for the airport, which involves a lot of analyses and predictions requiring huge amount of record and data to be collected, processed, analysed and archived in time. A proper information system not only helps keeping things in order and shortens the information flow cycle, but also helps to improve reliability, adds customer satisfaction, and saves a lot of paper to make the engineer process more environmental friendly.

One of the major drives in the Authority's geotechnical section for a better information system came in 1998 when the geotechnical section was re-structured and down sized to 1 engineer with 2 technicians taking over all the tasks and systems. The need for an efficient system had never been so urgent. System re-structure and integration were thus planned and implemented by stages.

The current geotechnical information system can be divided into four levels for information security control according to data type and output purpose as the following:

1. **Data / Operation** level- contains raw data entry modules, and output copies of original records. E.g., raw data obtained from an extensometer; borehole logging; laboratory tests, construction and maintenance records.
2. **Information / Monitoring** level- contains data translation modules, and output status/condition information. E.g. movement of each target in the extensometer; geological information aligned with ground sections; critical factors affecting ground condition in chronological order.
3. **Knowledge / Management** level- contains analytical modules, and output decision information. E.g. soil layer compression parameters; ground geological model; identified cause and action statements for the effect.

4. **Intelligence / Strategic** level- contains scenario analytical modules and output planning information. E.g. predicted timing and status of critical settlement; location and extent of critical settlement; maintenance plan and budget.

Level of computerisation in each of the above-mentioned levels is driven by demands and amount of information flow, with an ultimate aim of maintaining a highly competitive engineering efficiency. Continuous development is not always possible, as it is limited by available resource assigned to the system, and is limited by the level of technology available in the market as well. The following Table 3 outlines the past and future of the Authority's geotechnical information system. The future part is somewhat a projection to be adjusted according to the ever-changing world, but it provides a short-term direction for current development.

Table 3- Development trend of the Authority's geotechnical information system.

	Information system and Development		
	Past (Construction)	**Current (Operation, Maintenance & Development)**	**Future (Business Co-operation & Competition)**
System Development Emphasis	Ensure archiving and updating large amount of records in time, and make them retrievable for analysis. Develop data interpretation modules to speed up repetitive analysis cycles.	Simplify the structure of databases and application modules, and provide a standard user-friendly interface to reduce manpower requirement in application and maintenance. Ensure information reliability and security.	Ensure information in time, and achieve highly efficient information flow and analysis for particular business purposes. Develop an e-engineering system able to communicate with other major external systems.
Contents	• Design information, construction and as-constructed records • S.I. factual information • Survey records • Geotechnical and geological database • Instrumentation database	Additional information type: • Asset information • Operation and serviceability data • Defects records • Maintenance records • Ground movement trends • Scanned documents	Additional information type: • Defects warning • Product data • Cost data • Productivity data • External links • etc
Information Processing	Major manual, minor by computer.	About half by computer, and half manual.	Mostly by computer, minor manual.
Computer Applications	Ground investigation database, instrument database, survey database.	Geographical information system (relational database with map interface for user) integrating digitised document with other existing databases and analytical modules.	Distributed databases and application modules integrated by workflow applications with user portal and map interface.

Conclusion

Mission of geotechnical engineering in the context of airport operation has to assist the corporation to comply with all regulation requirements in airport maintenance and development. Instrumentation, being one of the major works continued from construction stage is discussed with updated information from operation stage. In addition, information system improvement and innovations has been as important as the engineering operation itself to keep the geotechnical engineering section slim but versatile and efficient.

References

Hendy M., Wong K.Y., and Luk M. (2002) "Hong Kong International Airport Platform at Chek Lap Kok- Ground Movement: Review and Prediction Using GIS" Paper accepted by *Geotechnical Aspects of Mass Transportation Systems*, HKIE Annual Geotechnical Seminar, Hong Kong.

Luk C.W.M. (2001). "Engineering for Airport Island Construction and Development- The new Hong Kong International Airport." *Proc., International Airport Symposium 2001*, Osaka, Japan, 2001, CD-ROM.

Plant G.W., Covil C.S., and Hughes R.A. (1998) edited. Airport Authority Hong Kong. *"Site Preparation for the New Hong Kong International Airport."* Thomas Telford Publishing Ltd., U.K.

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提高港海地质斟探效益新方法

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MAXIMIZATION OF BENEFITS IN GROUND INVESTIGATION FOR MASS TRANSPORTATION PROJECTS INVOLVING MAJOR COASTAL RECLAMATIONS IN HONG KONG

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撮要

要提高和運輸发展相关塡海工程的地质斟探的效益和准确性， 斟探的方法必须包括三方面的条件和考虑。 分述如下：

- 要以全球性、根据远洋地层资料为基础的地层作为模式，去阐释香港海域的地层分层。如严维枢(1994)指出， 香港海域地层存在五次冰期—间冰期证据，基本上可以和南極 VOSTOK 冰芯对比。
- 多利用鉆孔及鉆芯证据， 去强化同地区地震剖面的阐释， 包括受音浊现象影响地区。
- 加强对海泥鉆芯样本的地质和工程性质的探测。 这些探测技术， 包括如 CPT 等的常规工程测试， 和引进新的探测技术。我们购置的 GEOTEKMULTI-SENSORCORELOGGER， 可以直接探测鉆芯的 P波速、声阻抗值、密度、和磁化率等。 这些工程参数， 可以强化我们对港海泥土的工程性质的了解。

MAXIMIZATION OF BENEFITS IN GROUND INVESTIGATION FOR MASS TRANSPORTATION PROJECTS INVOLVING MAJOR COASTAL RECLAMATIONS IN HONG KONG

W. W. -S. Yim,[1] W. N. Ridley Thomas[2] and L. S. Chan[1]

ABSTRACT: A method for the maximization of benefits in ground investigation for mass transportation projects involving major coastal reclamations in Hong Kong is presented. This is made up of three parts:

(1) The adoption of an offshore geological model supported by global, regional and local evidence. In this regard, the model of Yim (1994) with up to five interglacial-glacial cycles in agreement with the record found in the Vostok ice core in Antarctica is chosen.

(2) Improvement in the interpretation of offshore seismic profiles including areas affected by acoustic turbidity aided by the use of boreholes for ground-truthing.

(3) Improvement in the profiling of engineering properties of offshore soils aided by field-testing using cone penetration and laboratory testing including the Geotek multi-sensor core logger. This logger permits the determination of p-wave velocity, bulk density, acoustic impedance, magnetic susceptibility, colour and natural gamma.

The method presented is suggested to have numerous benefits including a better understanding on the engineering properties of the offshore soils. Selected examples of mass transportation projects in the past will be used for illustration.

INTRODUCTION

An understanding of ground conditions and geological problems is at the forefront of issues in urban construction in the Hong Kong SAR (Page and Reels 2000). Unforeseen difficult ground conditions would result in the dramatic increase in cost of construction. This is particularly so in the case of coastal development because coastal reclamations are being extended further offshore into the continental shelf.

There are two common reasons for the high cost of large mass transportation projects involving major coastal reclamations in Hong Kong. First, because of the already developed nature of the coastline, the lack of alternative site options often means that there is little choice but to develop a site allocated at any cost. Second, the possible failure of the offshore ground investigation carried out to identify problems likely to be encountered at an early stage to provide warning. Two recent examples are the ground settlement problems of the Junk Bay new town and the foundation problems of the Tung Chung new town. Therefore there is a need to carry out ground investigation both comprehensively and cost-effectively. In the present paper, a method for the maximization of benefits in ground investigation carried out in providing the necessary information on the ground conditions is presented. The main objective is to assist construction and design.

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This paper is divided into four sections. Section one is on choosing an offshore geological model. Section two is on the interpretation of seismic profiles. Section three is on the profiling of engineering properties. Section four is a general discussion including the use of selected examples of previous mass transportation projects for illustration.

Choosing an Offshore Geological Model

For any coastal engineering project, choosing a valid offshore geological model is a matter of great importance. The model should not only account for the origin of the different types of deposits and their post-depositional changes but also for their engineering properties. Dating using a combination of methods including the stratigraphic succession, sedimentological properties, engineering properties, as well as global, regional and local correlation should be used to determine the validity of the model chosen.

Two offshore geological models are currently used in Hong Kong. The model of Yim (1994) subsequently modified by Yim and Choy (2000) and that of the Hong Kong Geological Survey (Fyfe et al. 2000). Some of the merits and limitations of the two models were discussed previously by Yim and Fyfe (1992) and Yim et al. (1998) while a possible correlation between the two models was given in Yim (1996). There are four main reasons why the model of Yim (1994) is preferred for use in Hong Kong. First, the Hang Hau Formation of Fyfe et al. (1997), named after the type locality in Junk Bay, is problematic. This formation was assigned an oxygen-isotope (OI) age of stage 1 (Holocene) and stage 2 (postglacial pre-Holocene). However, the study of borehole stratigraphy aided by radiocarbon dating (Yim 1999) has revealed the presence of a hiatus dated at ca. 8,100 yr BP between the M1 unit of OI stage 1 age and the T1 unit of OI stages 2 to 4 age (Yim and Choy 2000). The presence of this hiatus is in agreement with the timing of the 8,200 yr BP event when sea-level rose in response to the melting of the Laurentide ice cap (Barber et al. 1999). Second, at the type locality of the Chek Lap Kok Formation which was first proposed by Strange and Shaw (1986), supporting evidence for an older marine unit was found in a borehole (Yim and Li 1983). The fossils present in the older marine unit include foraminifers, ostracods, echinoids and molluscs. Furthermore the lower abundance or lack of marine remains in the M2 unit compared to the M1 unit is explained by their destruction during sub-aerial exposure of the deposits in OI stages 2 to 4 (Yim 1994; Yim and Choy 2001). The calcareous shells were destroyed partially or completely by groundwater dissolution associated with acid-sulphate soil development (Yim and Tovey 1995). Third, the model of Yim (1994) with five interglacial-glacial cycles is supported by agreement with the record found in the Vostok ice core in Antarctica (Petit et al. 1999) as well as regional evidence including loess stratigraphy (Yim 1996). Fourth, the model of Yim (1994) is supported by radiocarbon dating (Yim 1999), uranium-series dating (Yim et al. 1990; W.W.-S. Yim and M.T. McCulloch, unpublished work) and thermoluminescence dating (Yim et al. 2002a).

Table 1 provides a summary of the offshore geological model chosen for Hong Kong. Additional supporting evidence for this model includes the preservation of diatoms (Yim and Li 2000), the development of palaeosols formed by the sub-aerial exposure of marine deposits during glacial periods (Yim and Tovey 1995; Tovey and Yim 2002), and the application of the model in accounting for the offshore sand and gravel deposits exploited in a large scale during the 1990s (Yim 2001).

Interpretation of Seismic Profiles

Seismic profiling records are used in offshore ground investigation to provide a regional stratigraphical model of the transported and residual soils overlying the bedrock including information on their thickness and possible origin. The profiles are based on significant changes in acoustic impedance obtained using a constant seismic velocity of 1.6 km s^{-1} which if significant would give rise to distinctive reflectors. It is based on the appearance of reflectors that an interpretation is made aided by borehole logs, *in situ* cone penetration tests, and laboratory testing of borehole samples. Therefore boreholes are very important for ground-truthing the seismic profiles and it follows that a valid offshore geological model is needed to facilitate interpretation.

Table 1 Classification of Quaternary superficial seabed deposits in Hong Kong, their estimated ages, equivalent OI stages and estimated maximum thickness. Based on Yim (1994) and Yim and Choy (2000) with modifications.

Stratigraphic Unit*	Estimated age (yr BP)	Oxygen-isotope stage(s)	Age	Estimated maximum thickness (m)
M1	<8,100	1	Holocene/post glacial	21.5
T1	8,100-70,000	2-4	Last glacial	6.5
M2	90,000-140,000	5	Last interglacial	15.7
T2	150,000-180,000	6	2nd last glacial	9.5
M3	190,000-240,000	7	2nd last interglacial	13
T3	250,000-300,000	8	3rd last glacial	7.3
M4	310,000-340,000	9	3rd last interglacial	14.2
T4	350,000-370,000	10	4th last glacial	6
M5	380,000-420,000	11	4th last interglacial	5.5
T5	>440,000	12	pre-4th last interglacial	7

* M – marine; T – terrestrial.

The components of a high-resolution seismic profiling seismograph suitable for ground investigation in engineering are:

(1) A sound source having an initial pulse with an acceptable amplitude and a short rise time.
(2) A hydrophone streamer with low noise characteristics.
(3) A pre-set voltage amplifier.
(4) A signal processor with modules for system gain; initial gain, and, ramp 1 with delay, slope and maximum gain controls.
(5) Ramp 2 having either transmission or seabed triggered initiation and either linear or parabolic amplifier status.
(6) A swell filter for removing the effects of waves.
(7) A time varied band pass filter for removing unwanted noise.
(8) Stacking to cancel out noise and enhance signals.
(9) A seismic recorder with time delay and other functions.
(10) A differential global positioning system linked to a computer system to log data and to enable the vessel to be navigated down the required survey lines.

EGS (Asia) Limited has recently developed a completely new high-resolution boomer which can be hand-carried. The main features of this system are summarized in Table 2.

C-VIEW is a new, innovative, digital acquisition, processing and charting system that provides near real-time sub-bottom profiling, side-scan sonar and bathymetric solutions. This system has been designed as a simple PC-based platform from which to integrate recording, processing and interpretation of digital sub-bottom profiling, side-scan sonar and bathymetric data. The individual modules include:

(1) Playback or record.
(2) RS232 input from navigation; echo sounder; gyro; magnetometer; logging control; cable payout meter, and, underwater USBL.
(3) Band pass filter.
(4) Seabed tracking.
(5) Oscilloscope display and TVG.
(6) Image display.
(7) Plotter output.

Table 2 Main features of the new boomer system of EGS (Asia) Limited.

Unit	Details
Low voltage boomer sound source	Operates at 700 volts. No heavy power pack is needed because the condensers are installed on top of the boomer plate.
Hydrophone	Installed around the boomer plate edge.
Cables	There is only one multi-purpose cable attached to the boomer plate.
Low voltage pulse generator	Voltage range 400 to 700 volts. 620 volts has been found to be suitable in practice.
High power PC	This PC will process in-coming seismic signals from the hydrophone and prepare them for printing.
Printer	For printing the in-coming seismic signals.
Software	An in-house software package called C-VIEW.

These modules can be configured to provide six channels for data processing and display. Configuration data, including the window layout and all the processing parameters, are stored in parameter files, which can be saved, named and selected by the operator. This allows for standard and job specific configurations to be saved. Once the data has been acquired and stored by C-VIEW, data processing typically includes:

(1) Bandpass filtering.
(2) Bottom tracking for fish height calculation.
(3) Swell filtering.
(4) Wave rectification.
(5) Sequential shot stacking.
(6) Oscilloscope module for waveform display and TVG application.
(7) Slant range correction and X-Y coordination for mosaics.

The windows format provides an ideal platform from which to display the data. A holistic approach to data quality control and interpretation is provided through a multiple screen setup. In this way direct access to all survey data at any instant is possible.

The display module provides landscape or waterfall gray-scale scrolling. It also provides a user interface for interpretation functions including:

(1) Full data measurements of seabed features from side-scan data including target co-ordination (X-Y); target height; target along-track length; target across-track width; target annotation; distance between targets, and, seabed gradient.
(2) Full data measurement of sub-seabed features including up to eight reflectors; depth below the transducer; depth below the seabed to facilitate sediment profile calculations; digital terrain modeling; text annotation, and, data compression.
(3) The interpreted features are stored in ASCII files, which can be imported into ACAD.

An example of a seismic record generated by this new package and plotted with a simple printer is shown in Figure 1. The data has been interpreted and is ready for export directly into data display and plotting by Autocad. Although an interpretation can be made in the absence of boreholes for ground-truthing, the interpretation requires boreholes for verification.

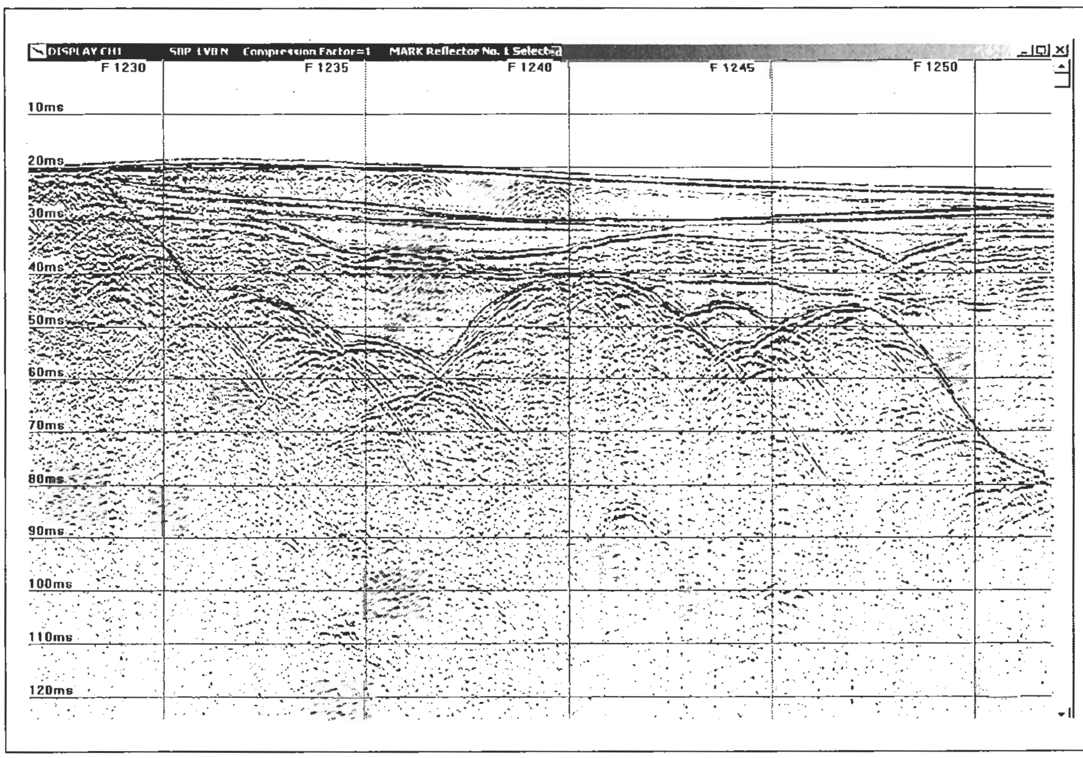

Figure 1 Example of a seismic record from northern Lantau Island near Yam O processed using C-VIEW and printed directly onto a standard ink-jet printer. Although an interpretation can be made (see solid lines inserted; the units from the base to the top – grade III moderately decomposed rock; grade V completely decomposed rock; pre-Holocene marine and terrestrial deposits, and Holocene M1 deposits), boreholes are needed for ground-truthing.

PROFILING OF ENGINEERING PROPERTIES

In order to fully understand the engineering characteristics of the sub-seabed materials encountered in coastal reclamations, the continuous profiling of their properties is needed. Previous work by Tovey (1992), Yim (1994), Yim and Choy (2000) and Tovey and Yim (2002) have revealed a close relationship between engineering properties of offshore soils and the geological history, namely the depositional and post-depositional environmental changes induced by Quaternary sea-level changes. Traditional methods such as *in situ* cone penetration testing first introduced for the ground investigation of the new Hong Kong International Airport during the late 1970s have proven to be extremely valuable and is now used routinely for the field-testing of localities of coastal reclamations (Paisley 1992). The benefits can be further enhanced by the use of other methods for profiling engineering properties.

The Geotek multi-sensor core logger was developed originally for deep-sea cores in the Ocean Drilling Program by Schultheiss and Weaver (1992). This logger is used as a tool for measuring a range of parameters including p-wave velocity, bulk density, magnetic susceptibility, colour and natural gamma. Typically cores in PVC casing can be logged non-destructively at a rate of 12 metres per hour and at sampling intervals of down to 1 mm. Based on the p-wave velocity and the bulk density obtained, the acoustic impedance can be determined to assist the ground-truthing of offshore seismic profiles by choosing the most appropriate seismic velocity. Additionally magnetic susceptibility and colour imaging provide means of identifying palaeo-desiccated crust formed by soil development on the sub-aerially exposed marine deposits (Yim and Tovey 1995; Tovey and Yim 2002). The information obtained together with cone penetration testing results would facilitate improvement in the interpretation of the engineering properties of the offshore soils. More recently, the logger was modified for measuring split cores to provide more reliable results (Gunn and Best 1998) while a calibration method for quantitative acoustic impedance studies was developed by Best and Gunn (1999). The main features of the Geotek multi-sensor core logger are summarized in Table 3.

Table 3 The main features of the Geotek multi-sensor core logger.

Unit	Details
p-wave	500 kHz piezo-electric ceramic transducers either arc or flat type accurate to 0.2%
Gamma ray attenuation (bulk density)	137-Cs gamma source in a lead shield. Density resolution of better than 1% depending upon count time.
Magnetic susceptibility	Bartington loop sensor 60 to 150 mm diameter or point sensor on split cores. Calibration accuracy 5%.
Calibrated colour core imaging system	Collects digital images using a line-scan camera linked to the core conveyor stepper motor to generate synchronous output of image data.
Natural gamma	Total natural gamma count or gamma spectra (K, U and Th) from two or more sodium iodide crystals.
Software	For logging cores, viewing graphical output and processing data in real time.

Apart from measuring cores using the Geotek multi-sensor core logger, if core samples are available for destructive testing, they should also be used as continuously as possible for testing index properties such as particle size, moisture content, Atterberg limits, etc. An example of an application is using moisture content to help confirm the presence of four marine units (M1, M2, M3 and M4) in a continuously sampled borehole from the new Hong Kong Airport site (Yim and Choy 2000; Yim et al. 2002a). The Holocene M1 unit was found to show the highest moisture content because it has never been sub-aerially exposed. On the other hand, the M2 to M4 units were found to show lower moisture contents with the lowest moisture content associated with the palaeo-desiccated crusts present in the upper part of these units. The profiling of engineering properties such as moisture content should therefore greatly enhance the understanding of the engineering properties of the offshore soils as well as the processes leading to their present state. Furthermore the palaeo-desiccated crusts are of greater soil strengths because of iron cementation through acid-sulphate soil development associated with the sub-aerial oxidation of the iron sulphides (Yim and Tovey 1995; Tovey and Yim 2002).

General Discussion

The offshore geological model chosen has already been applied successfully in a number of mass transportation projects involving coastal reclamations. These include the East

Harbour Crossing (Yim 1994), the Sheung Wan Station of the Island Line of the Mass Transit Railway Corporation (Yim 2000) and the new Hong Kong International Airport (Yim 1999; Yim et al. 2002a). In the East Harbour Crossing, the T2, T3, T4 units underlying the M1 and M4 units was suggested by Yim (2001) to form a natural sand foundation for immersed tunnel tubes. Adjacent to the Sheung Wan Station, cobble and boulder deposits underlying the M1 and M2 units were attributed to the T2 unit (Yim 2000). At the site of the new Hong Kong International Airport, the model helped to confirm the young age bias of radiocarbon dates from samples collected below the M1 unit (Yim 1999) while 237 moisture content measurements and thermoluminescence dating of a continuously sampled borehole helped to confirm the existence of the M1, M2, M3 and M4 units and the T3 and T4 units (Yim et al., 2002a).

The current practice of ground-truthing seismic profiles with boreholes to assist interpretation requires improvement. In particular the existing borehole logs produced for ground investigation needs to adopt the offshore geological model of Yim (1994) to facilitate interpretation. As a consequence of coastal reclamations extending progressively further offshore into the deeper parts of the continental shelf, the simple sequence of a single terrestrial unit overlain by a single marine unit (Holt 1962) is replaced by multiple marine and terrestrial units. Additionally, it is also desirable to perform p-wave and bulk density measurements on borehole samples so that acoustic impedance can be determined for ground-truthing the seismic profiles using an appropriate seismic velocity instead of a constant seismic velocity of 1.6 km $^{-1}$. Furthermore for high-resolution seismic interpretation, the reprocessing of seismic profiles using the best available seismic velocity according to the depth identified to be of interest is needed. This will ensure that the highest quality information is obtained from the profiles to assist ground investigation.

In addition to cone penetration testing, the profiling of engineering properties involving the use of the Geotek multi-sensor core logger and the testing of borehole samples is recommended for improving our understanding on the offshore soils. A better understanding of the cycle of events and natural processes that affect the engineering properties of the offshore soils in the manner described by Chandler (2000) is needed. It is by the profiling of engineering properties that light can be shed on the combined effects of deposition, consolidation/compaction, diagenesis, tectonic disturbance, weathering and erosion. Although there are few examples of case studies at present, features such as the existence of palaeo-desiccated crust formed by the sub-aerial exposure of marine deposits are supported by cone penetration testing (Endicott 1992), soil microfabric studies (Tovey and Yim 2002), moisture content profiling (Yim et al. 2002a) and magnetic susceptibility studies (Wong 1996). The availability of the Geotek multi-sensor core logger will assist the rapid advancement of this frontier of research.

One frontier of future research beneficial to coastal reclamations is the presence of gas in the offshore soils. The gas present may include carbon dioxide formed by the dissolution of calcareous marine fossil remains, hydrogen sulphide formed by sulphur-fixing bacteria and methane formed by the bacterial decay of plant matter. Existence of such gas trapped on the seabed is reflected through seismic profiles influenced by acoustic turbidity (Langford et al. 1995) and by the sampling and testing of such gas (Premchitt et al. 1992). Because of gas generation in the offshore soils, the lowering of soil strength may result in unforeseen ground settlement problems in future mass transportation projects involving coastal reclamations and detailed assessment is needed. In order to highlight such problematic localities, the use of seismic profiling for mapping areas of acoustic turbidity is a means of identifying such areas (Yim et al. 2002b). On the other hand, even in areas influenced by acoustic turbidity, the presence of trapped gas may provide valuable information on the stratigraphy of the seabed deposits as was demonstrated in Penny's Bay by Wong (1996).

CONCLUSIONS

Our recommendations for future mass transportation projects involving major coastal reclamations in Hong Kong if accepted should go some way in bringing the two disciplines of civil engineering and earth sciences closer together. This is seen to be a logical and much needed step forward because a good understanding of environmental change through time is needed to account for the engineering properties of the offshore soils. Through its adoption, improvements can take place through the maximization of ground conditions data obtainable through the geological and geotechnical investigations carried out both in the field and in the laboratory. This would help to facilitate the identification of difficult ground conditions at an early stage to permit contingency plans to be made thus reducing construction cost.

Future coastal development extending offshore further away from the present coastline is predicted to encounter new problems associated with the ground conditions. This includes not only more and more layers of marine deposits but also their greater cumulative thickness. Seabed areas influenced by acoustic turbidity may also be of concern because of the presence of trapped gases.

For future ground investigations, the use of the Geotek multi-sensor core logger for non-destructive profiling of p-wave velocity, bulk density, magnetic susceptibility, colour and natural gamma, and, the profiling of properties using borehole samples for destructive testing are suggested to be valuable tool. The determination of acoustic impedance in particular will help to improve the interpretation of seismic profiles as well as leading to a better understanding of the variability of soil strength in offshore soils provided through cone penetration testing.

REFERENCES

Barber, D. C., Dyke, A., Hillaire-Marcel, C., Jennings, A. E., Andrews, J. T., Kerwin, M. W., Bilodeau, G., McNeely, R., Southon, J., Morehead, M. D., and Gagnon, J.-M. (1999)." "Forcing of the cold event of 8,200 years ago by catastrophic drainage of Laurentide lakes. *Nature*, 400, 344-348.

Best, A. I., and Gunn, D. E. (1999). "Calibration of marine sediment core loggers for quantitative acoustic impedance studies." *Marine Geology*, 160, 137-146.

Chandler, R. J. (2000). "Clay sediments in depositional basins: the geotechnical cycle." *Quaterly Journal of Engineering Geology*, 33, 7-39.

Endicott, L. J. (1992). "Evidence from engineering properties." *The Logging and Interpretation of Transported Soils in Offshore Boreholes*, Yim, W.W.-S., Editor, Abstracts No. 8, Geological Society of Hong Kong and Department of Geography and Geology, The University of Hong Kong, Hong Kong, 39-48.

Fyfe, J. A., Selby, I. C., Shaw, R., James, J. W. C., and Evans, C. D. R. (1997). "Quaternary sea-level change on the continental shelf of Hong Kong." *Journal of Geological Society, London*, 154, 1031-1038.

Fyfe, J. A., Shaw, R., Campbell, S. D. G., Lai, K. W., and Kirk, P. A. (2000). *The Quaternary Geology of Hong Kong*. Hong Kong Geological Survey, Civil Engineering Department, The Government of the Hong Kong SAR.

Gunn, D. E., and Best, A. I. (1998). "A new nondestructive system for high resolution multi-sensor core logging of open sediment cores." *Geo-Marine Letters*, 18, 70-77.

Holt, J. K. (1962). "The soils of Hong Kong's coastal waters." Proc., Symposium on Hong Kong Soils, The University of Hong Kong, Hong Kong, 141-148.

Langford, R. L., James, J. W. C., Shaw, R., Campbell, S. D. G., Kirk, P. A., and Sewell, R. J. (1995). *Geology of Lantau District*. Hong Kong Geological Survey Memoir No. 6, Geotechnical Engineering Office, Hong Kong Government, Hong Kong.

Page, A. and Reels, S. J., Editors (2000). *The Urban Geology of Hong Kong*. Geological Society of Hong Kong Bulletin No. 6, Geological Society of Hong Kong and The University of Hong Kong, Hong Kong.

Paisley, J. M. (1992). "Evidence from cone penetration testing." *The Logging and Interpretation of Transported Soils in Offshore Boreholes*, Yim, W. W.-S., Editor, Abstracts No. 8, Geological Society of Hong Kong and Department of Geography and Geology, The University of Hong Kong, Hong Kong, 31-38.

Petit, J. R., Jouzel, J., Raynaud, D., Barkov, N. I., Barnola, J.-M., Basile, I., Bender, M., Chappellaz, J., Davis, M., Delamotte, M., Kotlyakov, V. M., Legrand, M., Lipenkov, V. Y., Lorius, C., Pepin, L., Ritz, C., Saltzman, E., and Stievenard, M. (1999). "Climate and atmospheric history of the past 420,000 years from the Vostok ice core." Nature, 399, 429-436.

Premchitt, J., Rad, N. S., To, P., Shaw, R., and James, J. W. C. (1992). "A study of gas in marine sediments in Hong Kong." *Continental Shelf Research*, 12, 1251-1264.

Schultheiss, P. J., and Weaver, P. P. E. (1992). "Milti-sensor core logging for science and industry." Proc., Oceans 92 – Mastering the Oceans through Technology, 2, 608-613.

Strange, P.J., and Shaw, R. (1986). *Geology of Hong Kong Island and Kowloon*. Hong Kong Geological Survey Memior No. 2, Geotechnical Control Office, Civil Engineering Services Department, Hong Kong Government, Hong Kong.

Tovey, N. K. (1992). "Evidence from soil microfabric and mineralogy." *The Logging and Interpretation of Transported Soils in Offshore Boreholes*, Yim, W. W.-S., Editor, Abstracts No. 8, Geological Society of Hong Kong and Department of Geography and Geology, The University of Hong Kong, Hong Kong, 49-60.

Tovey, N. K., and Yim, W. W.-S. (2002). "Desiccation of Late Quaternary inner shelf sediments of Hong Kong: microfabric observations." Accepted for publication in *Quaternary International*.

Wong, J. C. F. (1996). *Marine Seismic Interpretation of the Container Terminals 10 and 11 Site*. B.Sc. thesis, Department of Earth Sciences, The University of Hong Kong, Hong Kong.

Yim, W. W.-S. (1994). "Offshore Quaternary sediments and their engineering significance in Hong Kong." *Engineering Geology*, 37, 31-50.

Yim, W. W.-S. (1996). "Offshore Quaternary of Hong Kong: worldwide and regional correlation tables." *Quaternary Sciences*, 1996(3), 283-287 (in Chinese with English abstract).

Yim, W. W.-S. (1999). " Radiocarbon dating and the reconstruction of late Quaternary sea-level changes in Hong Kong." *Quaternary International*, 55, 77-91.

Yim, W. W.-S. (2000). "Drowned and partially drowned landslide deposits in Hong Kong." Proceedings Symposium on Slope Hazards and Their Prevention, 8-10 May 2000, The Jockey Club Research and Information Centre for Landslip Prevention and Land Development, The University of Hong Kong, Hong Kong, 78-83.

Yim, W. W.-S. (2001). "Stratigraphy of Quaternary offshore sand and gravel deposits in the Hong Kong SAR, China." *Quaternary International*, 82, 101-116.

Yim, W. W.-S., Chan, L. S., Hsieh, M., Philp, R. P., and Ridley Thomas, W. N. (2002b). " Carbon flux during the last interglacial cycle in the inner continental shelf of the

South China Sea off Hong Kong." Accepted for publication in *Global & Planetary Change*.

Yim, W. W.-S., and Choy, A. M. S. F. (2000). "Some engineering applications of Quaternary sea-level changes in Hong Kong." Proc., Conference on Engineering Geology Hong Kong 2002, The Institution of Mining & Metallurgy, London – Hong Kong Branch, Hong Kong, 211-218.

Yim, W. W.-S., Choy, A. M. S. F., Davis, A. M., and Huang, G. (1998). "Discussion on Quaternary sea-level change on the continental shelf of Hong Kong." *Journal of Geological Society, London*, 155, 1053-1055.

Yim, W. W.-S., and Fyfe, J. A. (1992). "Discussion on – towards a Quaternary stratigraphy for Hong Kong." *Geological Society of Hong Kong Newsletter*, 10(3), 19-23.

Yim, W. W.-S., Ivanovich, M., and Yu, K.-F. (1990). "Young age bias of radiocarbon dates in pre-Holocene marine deposits of Hong Kong and implications for Pleistocene stratigraphy." *Geo-Marine Letters*, 10, 165-172.

Yim, W. W.-S., and Li, J. (2000). "Diatom preservation in an inner continental shelf borehole from the South China Sea." *Journal of Asian Earth Sciences*, 18, 471-488.

Yim, W. W.-S., and Li, Q.-Y. (1983). "Sea level changes and sea-floor surficial deposits off Chek Lap Kok." *Geology of Surficial Deposits in Hong Kong*, Yim, W. W.-S., and Burnett, A. D., Editors, Abstracts No. 1, Geological Society of Hong Kong and Department of Geography and Geology, The University of Hong Kong, Hong Kong, 48-59.

Yim, W. W.-S., Price, D. M., and Choy, A. M. S. F. (2002a). "Distribution of moisture contents and thermoluminescence ages in an inner shelf borehole from the new Hong Kong International Airport." Accepted for publication in *Quaternary International*.

Yim, W. W.-S., and Tovey, N. K. (1995). "Desiccation of inner continental shelf sediments during Quaternary low sea-level stands." *Geoscientist*, 5(4), 34-35.

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Constitutive Model for Simulating Effects of Strain Rate on Strength and Elastic Moduli of Rocks under Uniaxial Compression

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Summary

A viscoelastic damage constitutive model is presented that simulates effects of strain rate on the strength and elastic moduli of rocks under uniaxial compression. The model is based on viscoelasticity and damage in the process of deformation of rocks. In the rheological portions of the model, multiple relaxation times can be obtained by adding elements, so we can simulate the effects of strain rate on the strength and elastic moduli in a wider strain rate range (strain rates ranging from about 10^{-7} to 10^{6}/s.). From the comparison of the model predictions with experimental data for porphyritic tonalite (at temperatures of 25°C, -78°C and -191°C) and sandstone (at room temperature), it is shown that the model can not only simulate the rate effects, but also can predict extremely sharp increase in the strength of rocks at about strain rate 10^{3}/s. Besides, the model can exhibit the upper and lower limits of the rate effects on the strength and elastic moduli.

摘要：基于岩石变形的粘弹性和损伤特性，建立一种能反映岩石在单轴压缩时强度和弹性模量的应变率效应。在模型的流变部分，通过增加有不同松弛时间的粘壶单元，可以在一个较宽的应变率范围内模拟强度和弹性模量的应变率效应（应变率范围约 10^{-7} to 10^{6}/s.）从模型的预测与英云闪长岩（温度为 25°C, -78°C and -191°C）和砂岩（室温）的实验结果的比较来看，该模型不仅能模拟上述的应变率效应，而且可以模拟应变率为 10^{3}/s.时的强度的急剧增加。此外，模型还可以模拟强度和弹性模量应变率效应的上下限。

1. Introduction

Knowledge of rock deformation and strength around an opening or tunnel is of great importance in engineering practice, and it will be of some value in studying stress wave spread and design in nuclear explosion and chemical explosion, thus a full constitutive curve is necessary for analysis (Pawel 1992). The effect of strain rate due to viscosity is common in cases of great depth, and for particularly susceptible rocks such as shales, sandstones etc. The various rock compression tests showed that both elastic moduli and strength are rate dependent quantities (Bieniaski 1970,Zheng 1992, et al.)

In tests (Perkin & Green 1970, Brace 1971, and Kumer 1993, et al.) on relatively isotropic and fine-grained geologic materials it is indicated that their strength and elastic moduli are strain rate sensitive, and that the strength increases with increasing the rate with going through a very sensitive region at about 10^{3}/s. However, a general theory model has not been established to express these rate effects quantitatively.

Osam (1981) considered theoretically a redistribution of microcracks due to subcritical crack growth and established equations to explain the effects of the axial strain rate on the strength and dilatancy, but his model do not exhibit the effects of the strain rate on elastic moduli. The damage constitutive model proposed by Cheng (1993) can simulate the complete stress-strain curves for whole strain range, but it can not simulate the effects of strain rate on strength and elastic moduli, let alone simulate sharp increase of the strength at strain rate about 10^3/s. Viscoelastic model or visco-elastoplastic model (Richard 1989,Tao 1991,et al.) can simulate the effects of strain rate on elastic moduli, but can not on strength.

In fact, in the process of the deformation for rocks, the rocks not only display rheological properties, but also damage properties, which may be regarded as enlarged microvoids, microcracks, or similar defects.

In this paper, a viscoelastic damage constitutive model is proposed by taking into account both viscoelasticity and damage. It can not only simulate the rate effects on the strength and elastic moduli for ordinary strain rate range ($\dot{\varepsilon} \leq 10^2$/s.), but also the sharp rate effects of the strength at about strain rate 10^3/s.; besides it can simulate both the upper and lower limits of the rate effects.

2. Theory Model

2.1 Viscoelastic Model

The deformation of rocks relating to time effect can be described by Boydin rheological model (Fig.1), which is also called the linear B type. The Boydin model can be equivalent to a generalized Kelvin model (Zheng 1995),which is also called the linear A type.

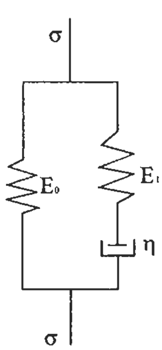

Fig.1 Boydin rheological model

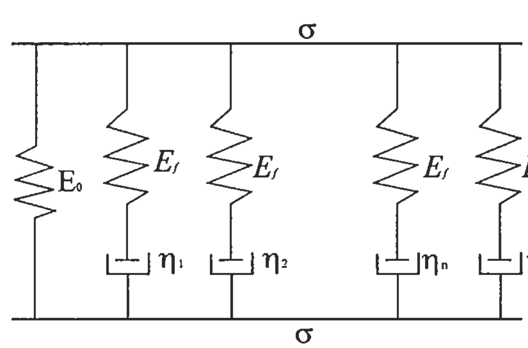

Fig.2 Improved rheological model

Differential equation of the Boydin model is

$$\dot{\sigma} + \frac{E_1}{\eta}\sigma = (E_0 + E_1)\dot{\varepsilon} + \frac{E_0 E_1}{\eta}\varepsilon \qquad (1)$$

where $\dot{\sigma}$ and $\dot{\varepsilon}$ denote the derivative of stress and strain with respect to time t, respectively. Suppose that the initial condition (shown as Fig.3) is $\varepsilon(0)$=0 and $\sigma(0)$=0 (t=+0), thus, $\sigma(t)$ in equation(1) is

$$\sigma(t)=E_0\varepsilon+E_1\int_0^t\dot{\varepsilon}(\tau)\cdot\exp(-\frac{t-\tau}{\theta_1})d\tau \qquad (2)$$

where $\theta_1=\eta/E_1$.

In the equation(2), damage is not considered.

2.2 damage

There is a large number of microvoids and microcracks within rock. This property can be described as inconsistent defects, including line defects, body defects and similar defects. The distribution of these defects is random and independent, and the average density of the defects is steady at any position within the materials. The distribution of the defects can be described using Poisson distribution

$$p(k/l)=\frac{(\lambda l)^k}{k!}\cdot\exp(-\lambda l) \qquad (3)$$

where λ stands for the mathematical expectation of the number of the defects of unit length; p for the probability of occurrence of k defects in length l.

In three-dimensional conditions, damage parameter is easily deduced (Tang 1993) as follows:

$$D=1-(\frac{1}{\alpha}+1)\cdot\exp(-\frac{x}{\alpha}) \qquad (4)$$

where $\alpha=1/\lambda$. x is determined by both the material properties and the sample shape.

In three-dimensional conditions, if there are three kinds of defects : line, surface and body defects, in general, damage paramete was

$$D=1-[(\frac{x}{\alpha})^m+1]\cdot\exp[-(\frac{x}{\alpha})^m] \qquad (5)$$

where m stands for dimensional number, it is determined by both the materials properties and the sample shape, α for the integrative parameter determined from the average distance of different defects within the material ,which is a function of external loading or deformation of material, and its dimension is length. The larger the loading is, the smaller α value. In simplest condition, we can consider that α is in inverse proportion to strain ε .

$$\alpha=\frac{c}{\varepsilon} \qquad (6)$$

where c is scale factor.

Suppose

$$\frac{x}{c}=\frac{1}{\varepsilon_0} \qquad (7)$$

It follows from equation(6), (7) and (5) that

$$D = 1 - [(\frac{\varepsilon}{\varepsilon_0})^m + 1] \cdot \exp[-(\frac{\varepsilon}{\varepsilon_0})^m] \qquad (8)$$

where m and ε_0 are the constants depending on both the properties and forms of the samples, ε strain when it increases. When strain decreases, D is constant which equals the maximum damage parameter D which it have reached to in loading history.

2.3 Viscoelastic damage constitutive model

For considering the time effects of the friction on the surface of the cracks and propagation of the cracks, which can be equivalent to defects development in the process of the deformation, first we develop a viscoelastic damage model which involves both viscoelasticity and damage under uniaxial compression. When the defects are under consideration, $\sigma(t)$ in equation(2) is replaced with $\frac{\sigma(t)}{(1-D)}$ (Lemaitre 1985),following equation will be obtained

$$\sigma(t) = (1-D)[E_0\varepsilon(t) + E_1 \int \dot{\varepsilon}(\tau) \cdot \exp(-\frac{t-\tau}{\theta_1})d\tau] \qquad (9)$$

where $$D = 1 - [(\frac{\varepsilon(t)}{\varepsilon_0})^m + 1] \cdot \exp[-(\frac{\varepsilon(t)}{\varepsilon_0})^m] \qquad (10)$$

In constant strain rate test, $\dot{\varepsilon}$=const. .In view of the initial conditions, there is the equality $\varepsilon = \dot{\varepsilon} \times t$,the solution of equation(9) is obtained:

$$\sigma(\varepsilon) = [(\frac{\varepsilon}{\varepsilon_0})^m + 1] \cdot e^{-(\frac{\varepsilon}{\varepsilon_0})^m} \cdot [E_0\varepsilon + \dot{\varepsilon}\eta(1 - e^{-\frac{E_1\varepsilon}{\eta\dot{\varepsilon}}})] \qquad (11)$$

Equation (11) can simulate stress-strain curves and the strain rate effects on the strength and elastic moduli in a narrower strain- rate range, which changes to about 10^3 times, but can not simulate the extremely sharp increase of the strength of rocks at about strain rate 10^3/s. .To avoid the disadvantage, the model equation(11) can then be improved by combining several Maxwell models with different parameters in parallel. The improved rheological model is shown in Fig.2, and its equation can be expressed as

$$\sigma(\varepsilon) = [(\frac{\varepsilon}{\varepsilon_0})^m + 1] \cdot e^{-(\frac{\varepsilon}{\varepsilon_0})^m} \cdot [E_0\varepsilon + \dot{\varepsilon}\sum_{i=1}^{n}\eta_i(1 - e^{-\frac{E_f\varepsilon}{\eta_i\dot{\varepsilon}}}) + \dot{\varepsilon}\eta_h(1 - e^{-\frac{E_h\varepsilon}{\eta_h\dot{\varepsilon}}})] \qquad (12)$$

By adding elements, multiple relaxation times can be obtained, due to the fact that the various defects have different time effects. So using the model we can simulate the effects of the strength and elastic moduli in a wider strain rate range.

To determine the parameters in equation(12), we can let $E_f = E_0\alpha_1 / n$, where n is determined by the strain rate range in which strength and elastic moduli increase with increasing strain rate, and the strength and elastic moduli increase about percentage α_1 per tenfold increase in strain rate. The increase of *n* can enlarge the rate-dependent range ; $\eta_i = E_f / c_i = \frac{E_0 \alpha_1}{n c_i}$, $c_i = 10^{i-4}$, its unit is s^{-1}. $E_h = \alpha_2 E_0$ and α_2 determined by the percentage which the strength increases sharply per tenfold increase in strain rate at about strain rate 10^3/s. . $\eta_h = E_h / c_2$ and $c_2 = 10^5$, its unit is s^{-1}. So equation(12) can be rewritten as

$$\sigma(\varepsilon) = [(\frac{\varepsilon}{\varepsilon_0})^m + 1] \cdot e^{-(\frac{\varepsilon}{\varepsilon_0})^m} \cdot [E_0 \varepsilon + \dot{\varepsilon} E_0 \cdot \frac{\alpha_1}{n} \sum_{i=1}^{n} \frac{1}{c_i} (1 - e^{-c_i \cdot \frac{\varepsilon}{\dot{\varepsilon}}}) + \dot{\varepsilon} E_h \cdot \frac{1}{c_2} (1 - e^{-c_2 \frac{\varepsilon}{\dot{\varepsilon}}})] \tag{13}$$

In equation(13),all variables will be in international system of units (SI units). The equation only can be used when strain rate is constant in the condition of uniaxial compression.

3. Comparison of Experiment and Model Prediction

The porphyritic tonalite was tested in uniaxial stress at strain rate from 10^{-4} to 10^3/s. at temperatures of 25°C,-78°C and -191°C . The material parameters,properties and the experimental techniques were described in detail by Perkins(1970) et al. . The sandstone was tested in uniaxial stress at strain rates from 3.3×10^{-5} to 3.3×10^{-3}/s. at room temperature by using DCS-Shimadzu testing machine. Right cylindrical specimens were drilled in the same direction from the same sandstone block.. The specimens' diameter was 19.2mm, and its length was 49.0mm. To avoid end effects, the ratio of length to diameter was fixed at 2.6 (Mogi, 1966). The specimens were tested under condition of dry state, and its porosity and density are 2.93% and 2.53g/cm^3, respectively.

In Fig.3,4 and 5, the experimental data for the tonalite and sandstone are compared with the curves predicted by model equation(13). Each experimental point represents the average of 4-8 test specimens. The fitted parameters, which are determined by the use of GA method(Genetic Algorithm)(Zheng,1999), are listed in Tab.1

Where, the elastic moduli are the secant moduli which are given by the slope of a line drawn from the origin to the point on the stress-strain cure corresponding to a 50% percent stress of the maximum stress or strength.

For the tonalite and sandstone at low and medium strain rate, the stress-strain theory curves predicted by the proposed model are in good agreement with the experimental results before the maximum stress, after the maximum stress the loading is uncontrolled, the strain rate increases and becomes so high that the stress and strain are not recorded, but in shape the model predictions (Fig.3)on stress and strain are similar to the typical experimental result recorded with servo-controlled system(Tang 1993,Pawel 1992).

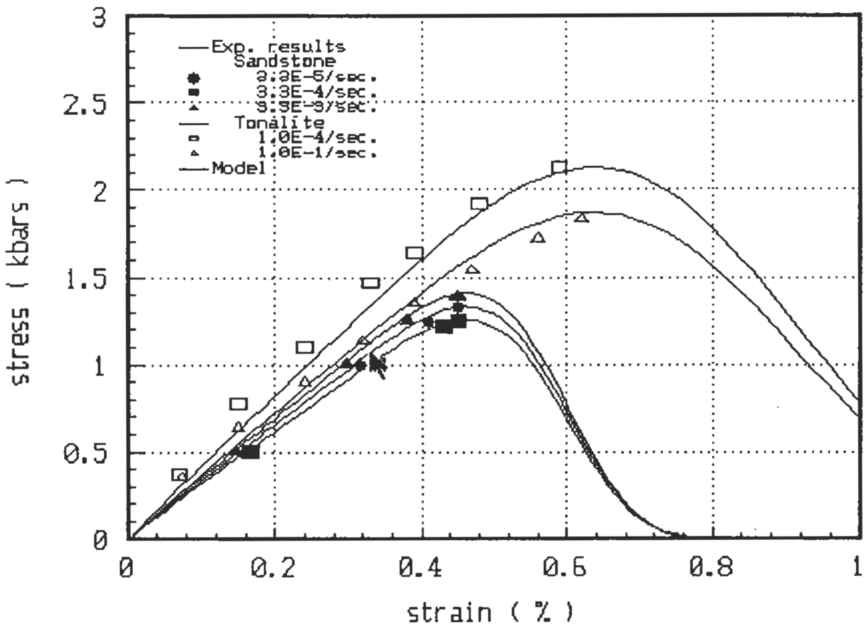

Fig.3 Comparison of experimental results and the models about stress and strain under uniaxial compression. (The tonalite after Perkin, the sandstone after Zheng)

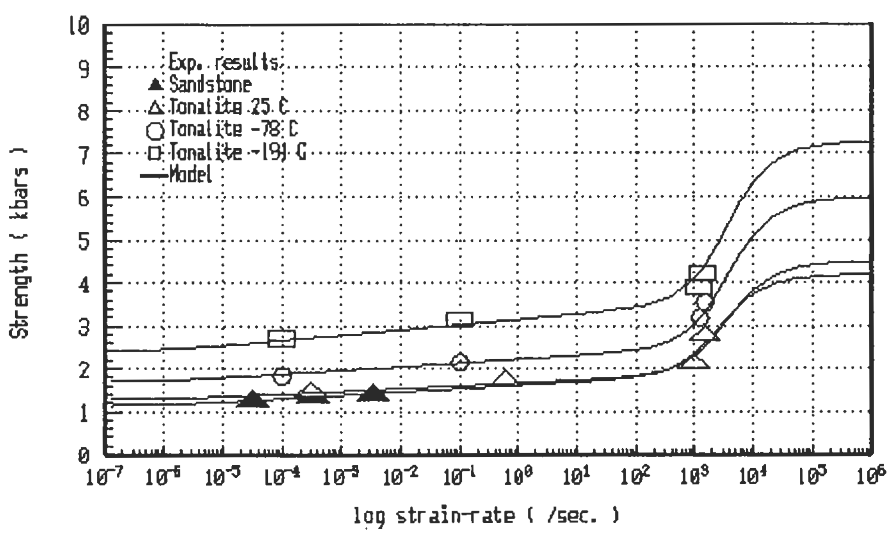

Fig.4 Comparison of experimental results and the model predictions about strength and strain rate under uniaxial compression

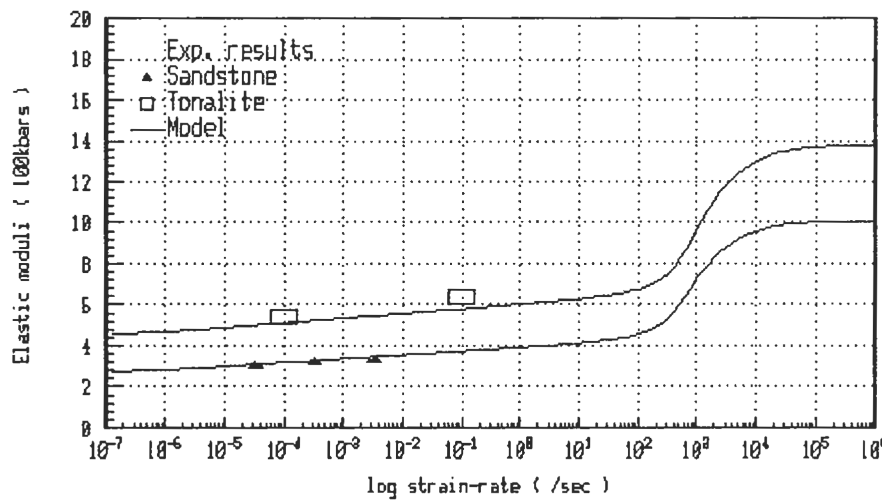

Fig.5 Comparison of experimental results and the models about elastic moduli and strain rate under uniaxial compression.

Note: the elastic moduli of tonalite are calculated from the data of Perkins.

Table 1. The fitted parameters for the tonalite and the sandstone

Rock Type	Test Temp.	E_0 (10^2kbar)	ε_0 ($\mu\varepsilon$)	m	n	α_1	α_2
Tonalite-	-190°C -78°C 25°C	2.43 3.24 4.60	7000	3.1	10	0.5	0.20
Sandstone	20°C	2.75	5200	5.2	10	0.67	0.21

The fig.4 shows that strength increases with strain rate at low and medium strain rate range, and behaves abrupt increase of rate-dependence near strain rate 10^3/s. ; the fig.5 shows that the strain rate effect on elastic moduli have a similar characteristic at the strain rate. The model predictions are in good agreement with these properties. At strain rate about 10^3/s., the experimental results of elastic moduli are very difficult to get exactly, but the model predictions show the very sensitive region (Fig.5). With the development of experiments technology, the predictions may be demonstrated in the future.

It is reasonable that the strain rate effects on the strength and elastic moduli of rocks shouldn't be infinite. This means that the effects of strain rate on the strength and the elastic moduli have a upper and lower limits. The model predictions(Fig.4 and Fig.5) can be in good agreement with the property.

At present our model is formulated for the uniaxial case. Its generalization to the case of the general stress state will be presented in a separate paper. Such a model could serve as a cornerstone to formulate complex theories proposed to evaluate the optimum progress of mining operation and to analyze long-time stability of the underground openings etc. .

During the deformation process, the rheological portions of the proposed model may be due to the time effects of the friction on the surface of the cracks and propagation of the cracks within the materials. For different cracks the time effects are different. The damage portion of the model may be due to the voids and cracks development, which can be equivalent to defects development described in defect parameter D.

4. Conclusion

In the paper, the proposed viscoelastic damage constitutive model can simulated the effects of strain rate on the deformation under uniaxial compression, include that of the strength and the elastic moduli of rocks, especially on the sharp increase of the strength at strain rate about 10^3 /s . . The model is in good agreement with the experimental results. Besides, it can show the upper and lower limits of the rate effects on the strength and elastic moduli.

References

Bieniawski,Z.T.(1966): Mechanism of rock fracture in compression, Rep.Counc. Scient. Ind. Res. S. Afr., MEG pp459.

Brace,W.F., and A.H. Jones (1971): Comparison of Uniaxial Deformation in Shock and Static Loading of Three Rocks, J. Geophys, Res., 76(20), pp4913-4920.

Cheng, H.B. and B.D. Maurice (1993): Deformation and Diffusion Behaviour in A Solid Experiencing Damage—A continuous Damage Model and Its Numerical Implementation. The 34th U.S. Symposium On Rock Mechanics June 27-30, pp495-498.

Kumer, M.P,and Paulo J.M.Monteiro (1993): CONCRETE structure, properties, and Materials, 2nd.ed. 456pp.

Lemaitre, J.A. (1985): Continuous Damage Mechanics Model for ductile Fracture, J. of Eng. Material & Technology, 107, pp83

Mogi, K.(1966):Some precise measurements of fracture strength of rocks under uniaxial compressive stress,Felsmech. Ingenieurgeol .,4, pp41-55.

OSAM, S.(1981): Influence of Strain Rate on Dilatancy and Strength of Oshima Granite Under Uniaxial Compression, J. Geophy.Res., 86, pp9299-9311.

Nawrocki, P.A. and Mroz. (1992): Constitutive model for rocks accounting for viscoplastic deformation and damage, Rock Mechanics ,Balkema. Rotterdam., pp691-700.

Perkin.R.D. and S.J.Green (1970): Uniaxial stress behaviour of porphyritic tonalite at strain rate to 10^3/second, Int. J. Rock Mech. Min.Sci.,7,pp527-535.

Goodman,R.E. (1989): Introduction to Rock Mechanics,2nd.ed. New York pp 204-217

Tang,C.A.(1993): Catastrophe in Rock Unstable Failure,(in chinese),Coal Industry Press,20pp.

Tao,Z.Y. and L.M. Zhang (1991):A Mathematical Model to Simulate the Stick-slip Mechanism of Faulted Rocks,Rock Mechanics as A Multidisciplinary Science, Reogiers(ed) , Balkman, Rotterdan, pp559-568.

Zheng,Y.L. & Xia S.Y.(1997): Mechanism of Dynamic Elastic Modulus of Geotechnical materials,16(2) (in chinese),Journal of Vibration and Shock, pp56-61.

Zheng,Y.L.(1992): An Investigation of Rock strain Rate Sensitivity and Dynamical Mechanics Properties,M.S.Thesis, Department of earth and space science,Univ. of Science and technology of China,Hefei, P.R.China.

Zheng,Y.L.(1999):Genic Algorithm and its Application of Modal Parameters of Structure Vibration, Journal of Tongji University,27 Suppl,pp102~106

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土釘加固填土坡的可靠度初步分析

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Preliminary Reliability Analysis of a Soil-nailed Fill Slope

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撮要

本文將響應面法(RSM)和電子表格法結合運用于土釘加固填土斜坡在排水條件下的可靠度分析。在可靠度分析中考慮了土容重，內摩擦角，護面沖剪強度，土釘黏結強度和屈服強度的不确定性。所進行的算例研究表明，土容重和內摩擦角對土坡的可靠度影響顯著，但是護面沖剪強度，土釘黏結強度和屈服強度對土坡的穩定性影響并不顯著。通過改變土釘間距，長度，傾角以及斜坡傾角這四個設計參數，可以得到可靠度指標的回歸分析模型。該模型可以用于求解設計由土釘加固的斜坡的可靠度或用于調整設計參數以達到指定的可靠度。

PRELIMINARY RELIABILITY ANALYSIS OF A SOIL-NAILED FILL SLOPE

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ABSTRACT: The reliability of a hypothetical soil-nailed fill slope in drained condition is analysed using a response surface method combined with a spreadsheet method. The failure mode considered is sliding failure, although other failure modes such as undrained static liquefaction of loose fill materials may also be very important. A limit equilibrium program SNALZ for soil nailing design is utilised as a deterministic model. Uncertainties associated with soil unit weight, soil friction angle, punching shear of nail facing, bond stress of soil-nail interface, and yield stress of nails are considered in the analyses. Variations of the probability of sliding failure of the fill slope with such design parameters as nail spacing, nail inclination, nail length and slope angle are investigated.

It is found that the reliability of the fill slope against sliding is influenced most significantly by the unit weight and friction angle of soil. On the other hand, the reliability is not sensitive to the uncertainties in the three nail parameters, i.e., punching shear, bond stress and yield stress. Based on the analyses, a regression model between the reliability index and the design parameters is obtained. This model can be used to estimate the reliability level of a soil-nailed slope with known design parameters and to generate a combination of design parameters that meet a specified reliability level of the slope.

INTRODUCTION

Many fill slopes were constructed in Hong Kong before 1977 by end-tipping fill materials, which typically consist of decomposed granite or volcanic, with little effort of compaction. Over the years, there have been a number of loose fill slope failures. One catastrophic failure was the 1976 Sau Mau Ping landslide (Hong Kong Government 1977), which blocked the ground floor of a residential building and killed eighteen people.

Recompaction of the top 3 m of fill materials has been the recommended way to update substandard loose fill slopes for over two decades. However, this traditional approach may be unpractical and unsafe to carry out in many congested and hilly sites. A possible alternative to stabilize those loose fill slopes is soil nailing.

Soil nailing is a soil reinforcement technique for supporting excavation and stabilizing slopes. A typical construction procedure (Byrne et al. 1996) is to: (1) drill holes in excavation or slopes; (2) install steel bars called nails into the slopes or excavation; (3) grout the holes; (4) place drainage strips, initial shotcrete layer and install bearing plates or nuts; and (5)

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repeat the above process to the final grade and install the final facing. Figure 1 shows a typical soil-nailed slope. The fundamental mechanism of soil nailing is the development of tensile forces in the soil nails as the result of the restraint that the reinforcements and the attached facing offer to lateral deformations of the structure (see Figure 2). The tensile force developed in the nails can be calculated according to the nail support diagram (Figure 3).

Little study has been devoted to probabilistic or reliability evaluation of soil nailing reinforced slopes. Kay and Li (1999) proposed a procedure to assess the probability of failure of nailed slopes in Hong Kong. The approach considered infinite slopes (translational failure mode) and focused on the uncertainties in rainfall. It assumed that the effect of soil nailing is to force any failure that might occur subsequent to the soil nailing process to occur on a failure plane that has a greater depth than the critical one prior to soil nailing. Combining the historical information on the performance of a large number of cut slopes in Hong Kong and the frequency data for different rainfall periods, the probabilities of failure of the soil nailed slopes were determined.

This paper presents the results of a preliminary study that aims at investigating the reliability of nailed loose fill slopes in drained condition considering soil-nail interactions. The stability of hypothetical nailed slopes (30 m high and varied slope angles) was evaluated by a limit equilibrium slope stability program SNAILZ that was developed by CALTRANS (1999). Then a response surface method (RSM) and a spreadsheet reliability analysis method were applied to calculate the reliability of the slopes. The influences of five random variables (soil unit weight, soil friction angle, punching shear of nail facing, bond stress of soil-nail interface, and yield stress of nails) on the reliability of the soil nailed slopes are studied. Variations of the probability of failure of the soil-nailed fill slopes with such design parameters as nail spacing, nail length, nail inclination and slope angle are investigated. The preliminary findings are presented and a regression model is proposed to obtain the corresponding reliability level.

Note that the use of soil nailing in loose fill slopes may involve such possible failure mechanism as static liquefaction of loose fill materials and rainfall infiltration is one of the triggers. Research on the stability of soil-nailed loose fill slopes due to static liquefaction and the probabilities of failure under different rainfall conditions is underway, undertaken jointly by the Hong Kong University of Science and Technology and the University of Hong Kong. In this paper, only the shear failure in drained condition is considered.

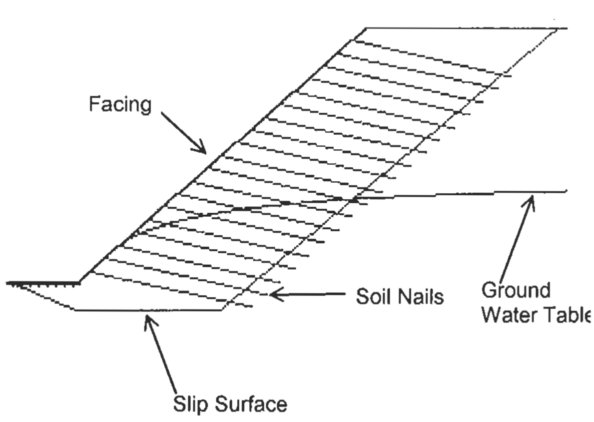

Figure 1. A Soil-nailed Slope

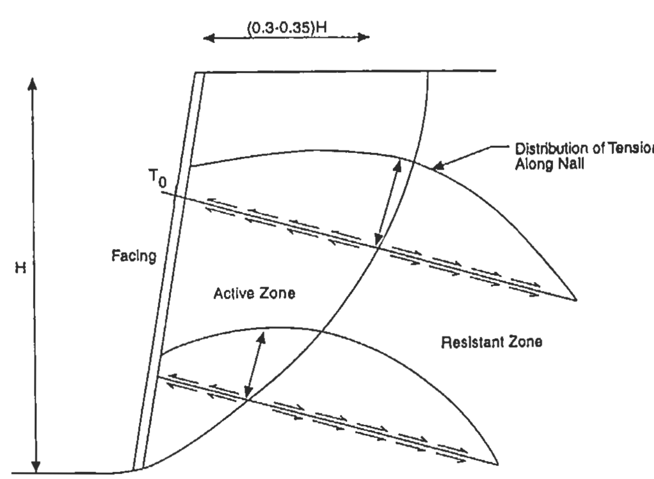

Figure 2. Schematic View of Soil Nail Behavior (Byrne et al. 1996)

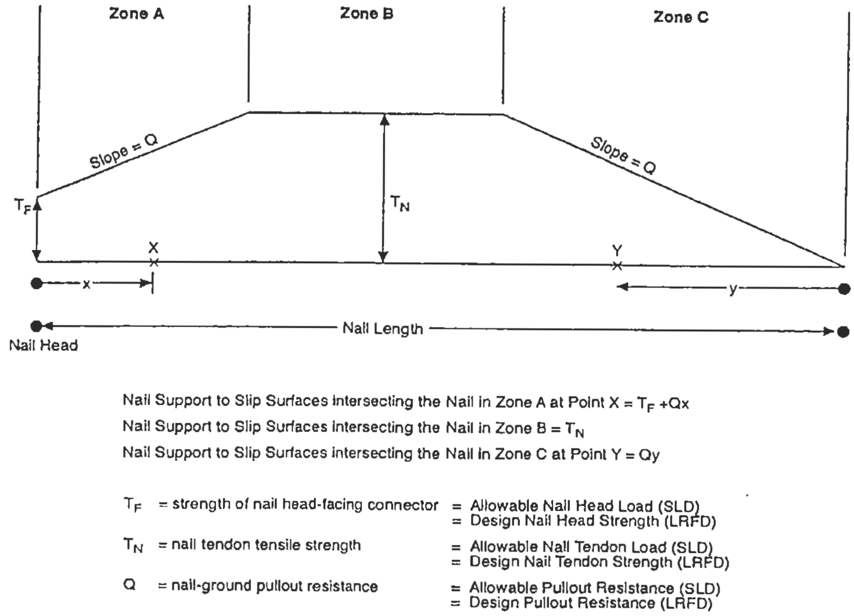

Figure 3. Nail Support Diagram (Byrne et al. 1996)

RELIABILITY ANALYSIS METHODOLOGY

For complex geotechnical problems such as slope stability problem, a limit equilibrium program or a finite element program is usually required and in many cases, the safety factor cannot be expressed as an explicit function of random variables. Approximate methods are needed to perform reliability analysis in such conditions. The response surface method (RSM) has been shown to be an efficient and practical method for reliability analysis (Bucher and Bourgund 1990, Faravelli 1989, Wong 1985) where performance functions cannot be expressed explicitly. The basic concept of RSM for reliability analysis is to approximate the implicit performance function $g(\mathbf{X})$, which defines the boundary between safe and failure regions, by a response surface function $g'(\mathbf{X})$. A sufficient number of experiments are conducted at sampling points in order to determine the approximated function and obtain the corresponding reliability. Iteration may be necessary if the requirement of acceptable error of reliability estimation needs to be fulfilled.

Usually the response function is a polynomial function of random variables. In the literature, second-order polynomial functions with cross-product terms or squared terms (Wong 1985, Bucher and Bourgund 1990) and linear polynomial function (Kim and Na 1997) have been proposed as the response surface function. In this study the adopted response surface function is the same as the one proposed by Bucher and Bourgund (1990):

$$g'(\mathbf{X}) = a + \sum_{i=1}^{n} b_i X_i + \sum_{i=1}^{n} c_i X_i^2 \tag{1}$$

where $\mathbf{X}$ =random variable vector; $g'(\mathbf{X})$ =response surface function; X_i = ith random variable (see Table 1); n = number of random variables; a, b_i and c_i = coefficients to be determined.

The performance function is defined as $g(\mathbf{X}) = SF(\mathbf{X}) - 1$, where $SF(\mathbf{X})$ is the safety factor given the random variable vector $\mathbf{X}$. Thus a point on the response surface $g(\mathbf{X})$ can be obtained by one slope stability analysis. By regression analysis, the coefficients of the above response surface function can be determined once enough safety factors are obtained.

After an approximated performance function is obtained, the reliability analysis methods can be applied to obtain the reliability index β. In this paper, the spreadsheet method proposed by Low (1996) and Low and Tang (1997) is utilized. This method belongs to the advanced first-order second-moment method (AFOSM). The spreadsheet tool such as MS-

Excel can be used to calculate the reliability index without much computational efforts. The details of the spreadsheet method can be referred to Low and Tang (1997).

The procedure of reliability analysis in this study is as follows:

1. In the first iteration, the mean value vector is chosen as the first center point. A set of sampling points $\mu_i \pm h_i\sigma_i$ (h_i is an arbitrary factor and is taken to be h_i =1.0 in this study; σ_i is the standard deviation) and the center point (a total of 2n+1 sampling points) are used to calculate the safety factor SF. By using the 2n+1 response values in regression analysis, the response function in Equation 1 can be uniquely determined.
2. After the g'(**X**) is obtained, the design point $\mathbf{X}_D$ is determined by using Solver tool in Excel.
3. The $\mathbf{X}_D$ is chosen as the new center point and new experiments are conducted at $\mathbf{X}_D$ and $\mathbf{X}_D \pm h_i\sigma_i$.
4. Steps 2-3 are repeated until β converges (tolerance level 0.001).

RESULTS AND ANALYSIS

In this study, the soil-nailing program SNAILZ was used as a deterministic model for stability analysis of the hypothetical fill slope reinforced with soil nails. The program uses a

Table 1. Statistics of Random Variables

Parameters		Mean	C.O.V (%)	σ	References
Unit Weight of Soil γ_{sat} (kg/m^3)	X_1	18	5	0.9	Liang et al. (1999)
Friction Angle of Soil tanϕ'	X_2	tan37°	15	0.113	Fung (2001)
Punching Shear of Facing (kN)	X_3	165	20	33	Assumed
Bond Stress of the Soil-Nail Interaction (kPa)	X_4	62.5	10	6.25	Byrne et al. (1996)
Yield Stress of Nail (kPa)	X_5	420	8	33.6	Val et al. (1997), Marek (1995)

Table 2. Parameters in the Parametric Studies

Nail Spacing (m)	Nail Inclination (degree)	Slope Inclination (degree)	Nail Length (m)	Nail Diameter (mm)	Drill Hole Diameter (mm)
1.0 1.5 2.0	15	35	18	32	100
1.5	10 15 20	35	18	32	100
1.5	15	25 35 45	18	32	100
1.5	15	35	8 12 18 24	32	100

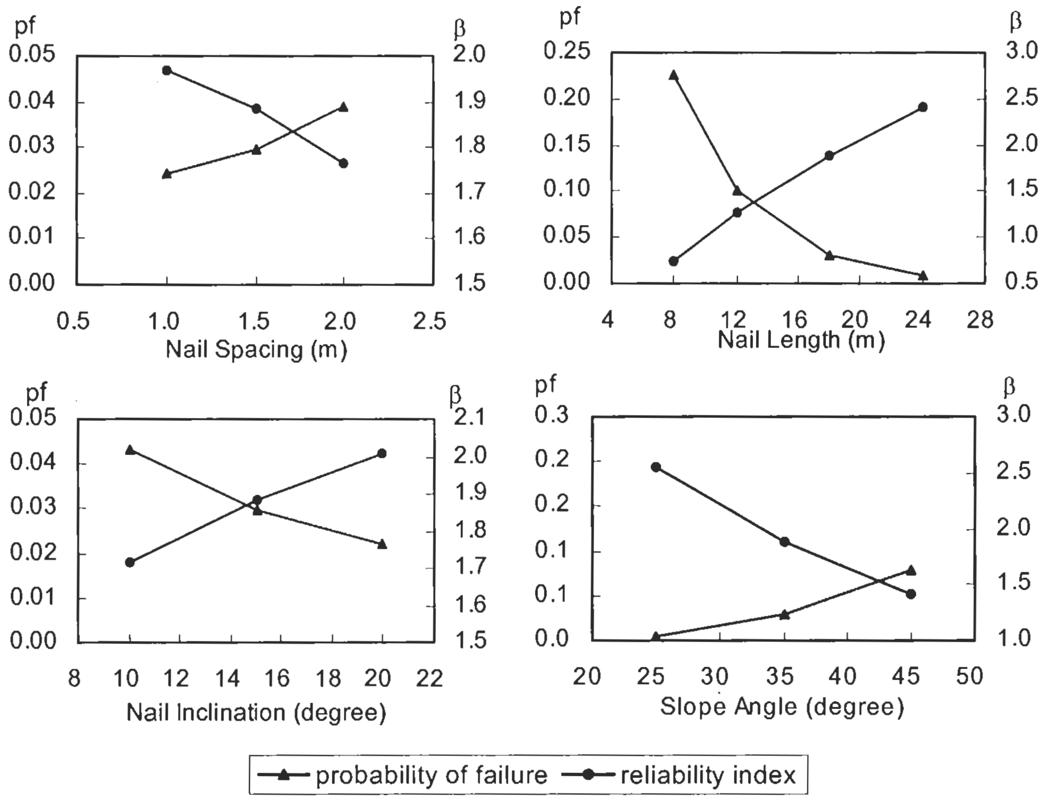

Figure 4. Influence of Nail Spacing, Nail Length, Nail Inclination and Slope Angle on Probability of Failure of Nailed Slope. See Table 2 for Combinations of Parameters.

bilinear or tri-linear wedge analysis for determining the minimum safety factor with interslice forces included.

The hypothetical slope is 30 m high. The water table is at 1/3 height of the slope. The five random variables, i.e., the saturated unit weight of the soil, soil friction angle, punching shear of the nail facing, bond stress of the soil-nail interface, and yield stress of the nails, with their statistics, are summarized in Table 1. It is assumed that all random variables follow the normal distribution and are statistically independent. According to Law et al. (1998), most loose fill materials forming the slopes before recompaction are characterized by an almost cohesionless behaviour (c'=0) for the saturated condition. Thus, the cohesion of the soil is assumed to be zero in this study. Totally there are ten parametric runs of reliability analysis in which nail spacing, nail inclination, nail length and slope angle are varied. The parameters for the parametric studies are listed in Table 2.

Table 3 shows the iteration results for the 35° slope with nail spacing of 1.5 m, nail length of 18 m and nail inclination of 15°. It is found that the coefficients related with punching shear, bond stress and yield stress X_3, X_4, X_5 are extremely small compared with the coefficients related with X_1 and X_2. It implies that strengthening soil nails may not produce significant improvement on the stability of a nailed slope. On the other hand, the unit weight and friction angle of the soil are very sensitive to the stability of the slope.

The effects of varying nail spacing (1.0 m, 1.5 m and 2.0 m), nail length (8 m, 12 m, 18 m and 24 m), nail inclination (10°, 15° and 20°) and slope angle (25°, 35° and 45°) on the slope reliability are investigated (Figure 4). Based on the results, a linear regression equation for the reliability index β is obtained

$$\beta = 1.85597 - 0.205 Z_1 + 0.0293 Z_2 + 0.10511 Z_3 - 0.05715 Z_4 \qquad (2)$$

Table 3. Example Iteration Results (nail spacing 1.5m, nail length 18 m, nail inclination 15°, slope height 30 m and slope angle 35°)

Iteration		1	2	3	4	5
Response Surface Function Coefficients	a	-0.8901	-5.4537	4.5633	-4.0000	-3.8111
	b_1	0.0222	0.2140	-0.6713	0.2384	0.2405
	b_2	1.6244	10.81589	1.7660	4.2358	3.4905
	b_3	-3.48E-13	2.06E-12	-8.56E-13	-2.28E-12	3.65E-12
	b_4	-0.0151	2.38E-11	0.0168	-2.09E-11	4.29E-11
	b_5	-1.08E-12	6.95E-12	-2.95E-12	-6.91E-12	1.051E-11
	c_1	2.41E-12	-0.00555	0.0185	-0.0061	-0.0063
	c_2	0.0930	-8.4760	1.305E-10	-2.1356	-1.4159
	c_3	1.056E-15	-6.20E-15	2.587E-15	6.91E-15	-1.11E-14
	c_4	0.000127	-1.90E-13	-0.000128	1.673E-13	-3.43E-13
	c_5	1.28E-15	-8.30E-15	3.50E-15	8.23E-15	-1.25E-14
β		1.713	1.7226	1.7246	1.716	1.716
Design point	X_1	17.842	18.025	18.036	17.876	17.893
	X_2	0.5609	0.7172	0.5587	0.5603	0.5601
	X_3	165	165	165.	165	165
	X_4	62.494	62.5	62.495	62.5	62.5
	X_5	420	420	420	420	420

in which Z_1 = nail spacing, Z_2 = nail inclination, Z_3 = nail length and Z_4 = slope angle. The standard error of this regression equation is 0.091.

Comparing the coefficients in the regression model and the results in Figure 4, it is found that as expected, the most efficient way to increase the reliability of a soil-nailed slope is perhaps to increase nail length. However, an increase of the nail inclination may also be a good way to improve the reliability level of a nailed slope. A very small nail inclination as adopted in a steep cut slope may not be suitable for fill slopes.

By using this regression equation, the reliability of a 30 m soil-nailed slope with given design configuration can be estimated. It may also be useful in optimizing the combination of design parameters such nail spacing, length and inclination in order to fulfill a specified reliability level.

Note that rainfall effect has not been considered and the ground water table is specified in the above analyses. The uncertainty of rainfall infiltration will be included in later study.

CONCLUSIONS

In this paper, the response surface method is used to perform reliability analysis of hypothetical soil-nailed fill slopes. The spreadsheet method is utilized to reduce computational efforts for obtaining the reliability index. An iteration procedure is applied to obtain the reliability index and an approximated limit state function is yielded for each parametric run.

From the results of parametric study, a linear regression model between the reliability index and several design parameters, i.e., nail spacing, nail length, nail inclination and slope angle, is obtained. For a soil nailed fill slope with a given design configuration, the corresponding reliability of the slope against sliding can be evaluated according the regression model. If a specified reliability level should be satisfied in soil nailed slope design, a set of optimized design parameters can be calculated.

The unit weight and friction angle of the soil are very sensitive to the stability of the slope. However, the punching shear of the facing, and the bond stress and yield stress of the nails are less sensitive to the slope stability. As expected, the reliability of the nailed slope increases with increased nail length and reduced nail spacing. An increase in the nail inclination may also contribute to the reliability of the slope.

References

Bucher, C. G. and Bourgund, U. (1990). "A fast and efficient response surface approach for structural reliability problems." *Structural Safety*, 7(1), 57-66.

Byrne, R. J., Cotton, D., Porterfield, J., Wolschlag, C. and Ueblacker, G. (1996). *Manual for Design and Construction Monitoring of Soil Nail Walls*, FHWA-SA-96-069.

CALTRANS (1999). *SNAILZ User Manual*, California Department of Transportation, USA.

Faravelli, L. (1989). "Response-surface approach for reliability analysis." *Journal of Engineering Mechanics*, ASCE, 115(12), 2763-2781.

Fung, W. T. (2001). *Experimental Study and Centrifuge Modeling of Loose Fill Slope*, M. Phil. Thesis, The Hong Kong University of Science and Technology.

Hong Kong Government (1977). *Report on the Slope Failures at Sau Mau Ping, August 1976*. Hong Kong Government Printer, Hong Kong.

Kay, J. N. and Li, K. S. (1999). "A probabilistic approach to soil nailing for Hong Kong slopes." *Geotechnical Risk Management, Proceedings of the 18th Annual Seminar Organized by the Geotechnical Division of the Hong Kong Institution of Engineers*, 79-85.

Kim, S.-H. and Na, S.-W. (1997). "Response surface method using vector projected sampling points." *Structural Safety*, 19(1), 3-19.

Law, K. T., Lee, C. F., Luan, M. T., Chen, H. and Ma, X. (1999). *Appraisal of Performance of Recompacted Loose Fill Slopes*, GEO Report No. 58, Geotechnical Engineering Office, Civil Engineering Department, Hong Kong SAR.

Liang, R. Y., Nusier, O. K., Malkawi, A. H. (1999). "A reliability based approach for evaluating the slope stability of embankment dams." *Engineering Geology*, 54(3), 271-285.

Low, B. K. (1996). "Practical probabilistic approach using spreadsheet." ASCE Geotechnical Special Publication, No. 58, *Proc., Uncertainty in the Geologic Environment-From Theory to Practice*, Madison, Wisconsin, July 31-Augest 3, Vol. 2, 1284-1302.

Low, B. K., Tang, W. H. (1997). "Efficient reliability evaluation using spreadsheet." *Journal of Engineering Mechanics*, ASCE, 123(7), pp749-752.

Marek, P. (1995). "Application in steel structures." *Probabilistic Structural Mechanics Handbook*, C. (Raj) Sundarajan (ed), Chapman & Hall, New York, USA, pp650-662.

Val, D., Bljuger, F. and Yankelevsky, D. (1997). "Reliability evaluation in nonlinear analysis of reinforced concrete structures." *Structural Safety,* 19(2), 203-217.

Wong, F. S. (1985). "Slope reliability and response surface method." *Journal of Geotechnical Engineering*, ASCE, 111(1), 32-53.

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