Mass Transit Railway Hong Kong - Geotechnical Aspects

J. A. Davis

1.0 INTRODUCTION

The Mass Transit Railway in Hong Kong began construction of its first line in 1976 with the Modified Initial System (MIS) which opened in 1980. This was immediately followed by the construction in 1978 of the Tsuen Wan Extension which opened in 1982. (see Figure 1)

In 1982 the construction of the Island Line begun which opened in 1986. (see Figure 1)

This year sees the start of the construction of the Airport Lantau Railway which as its name suggests links the new airport at Chek Lap Kok to the existing system. (see Figure 2)

This paper addresses some of the geotechnical aspects of design and construction related to each of the lines. In particular it will deal with some aspect on the geotechnical problems on the MIS, the Island Line Extension and some of the design aspects of the line to the Airport at Chek Lap Kok.

There have been a large number of papers published on the existing railway lines. For detailed of each of the aspects discussed reference should be made to the original papers.

2.0 Modified Initial System

The modified initial system involved the construction of 12km of underground railway together with 4km of viaduct structures.

As part of the construction two stations were to be constructed in the Central Business District of Hong Kong.

The two stations were a contrast in construction techniques in that space was available for open cut on the Admiralty Station site whilst Chater Station was constructed adjacent to existing high rise structure between heavily reinforced diaphragm walls.

Chater Station

Prior to the start of construction of the Hong Kong Mass Transit Railway in 1976, very little experience existed concerning the construction of diaphragm walls in residual soils. Previous published experience was mainly concerned with diaphragm wall constructed in sedimentary soils and, in most cases, attention was focused on the problem of ensuring the stability of the slurry filled trench (e.g. Dibiagio and Myrvoll, 1972; Huder, 1972) and movements although the few measurements made during full scale installations generally showed that, provided the stability of the trench was maintained, ground movements were fairly small (e.g. Cunningham and Fernandez, 1972; Karlsrud, 1975 and Burland and Hancock, 1977). Experience in Hong Kong however, has shown that for walls constructed in decomposed granite, significant ground movements can occur which can result in large surface settlements extending a considerable distance from the wall.

The major station to be built at Chater was within the Central Business Area of Hong Kong adjacent to very high rise structures. Figure 2 shows the location of the station and the section of wall to be discussed is shown on plan in Figure 3 and in section Figure 4. The typical soil profile is shown in Figure 5.

The diaphragm wall was 1.2m thick and up to 37m deep. The wall was constructed in the normal manner and the slurry level maintained within about 0.5m of ground level. This level correspond to a slurry head of around 1.5 to 2m above the general pre-construction groundwater level.

Towards the end of 1976, construction commenced on the north wall of the station near the Hong Kong Club. The progress of wall construction is shown in Figure 6. The wall came within about 5m of the Hong Kong Club and, as it passed, the building settlements of about 30mm were observed as shown in Figure 6. From this figure, it can be seen that settlements associated with the construction of individual panels were fairly small however, the settlement increased as adjacent panels were constructed.

Shortly after completion of the north wall, construction of the south wall commenced and, in April 1977, the wall passed the Courts of Justice to the south of the station. During this period, settlements of the building were measured both close to and at various distances from the wall. Close to the wall, settlements up to 80mm were observed on the eastern facade of the building decreasing more or less linearly to about 8mm at a distance of 50m (Figure 7). As found at the Hong Kong Club, the large settlements were a result of the cumulative effect of constructing a series of panels rather than the effect of a single excavation, as shown in Figure 8.

In subsequent investigations it was also found that the piezometric pressure in the decomposed granite to the south of the station had risen by about 1m due to the obstruction of water flow from the hills to the south as a result of construction of the diaphragm wall. Thus, during construction of the south wall near the Courts of Justice, the slurry head in the trenches above the groundwater level was probably about 1m less than that previously used in the construction of the north wall.

To examine the cause of the settlements, comparison was made between the observed movements and the ground deformations which had been measured adjacent to a single instrumented test panel excavation. The location of the test panel is shown in Figure 3 and the details of the excavation and instrumentation in Figure 9.

The instrumentation comprised of four inclinometers to measure horizontal ground movements adjacent to the trench and various surface survey points to monitor both settlement and horizontal movement at ground level. Settlements at depth were measured using deep settlements points and extensometer magnets fitted around the inclinometer tubes. Pore pressure changes during excavation were monitored using six pneumatic piezometers having almost instantaneous response.

The slurry level in the trench was then lowered in stages until collapse of the trench occurred. As the slurry level was lowered, surface settlements and horizontal movements of the ground increased as shown in Figure 11. Both surface settlements and horizontal movements in the fill and marine deposits were, however, still fairly small until the slurry level came within about 0.4m of the water table (+2.6mP.D.) when an increase in movement was observed. In the decomposed granite, the horizontal movements gradually increased until the slurry level was lowered to about 0.6m below the water table (+1.63m P.D.) when readings were stopped as collapse of the upper part of the trench started to occur. At this stage horizontal movements of up to 70mm had been measured in the decomposed granite at 1m from the excavation. When collapse finally occurred, the upper 5m of soil slid into the trench creating a cavern about 3.5m wide on one side of the trench.

It is clear from the measurements made during the test panel that the large settlements observed during the diaphragm wall installation were the result of the horizontal displacement in the decomposed granite during the excavation of individual panels.

In order to understand the settlements during diaphragm wall installation it is necessary to consider in detail the stress changes and displacements which take place in the ground during panel excavation, concrete placing and during the construction of adjacent panels.

During the excavation of a panel the horizontal total stress in the ground adjacent to the panel is reduced from the initial at rest pressure to the slurry pressure. This reduction of pressure provides a potential for the ground to swell. The magnitude of the displacement of the soil adjacent to the trench will depend on the swelling characteristics of the soil, the net slurry pressure, the panel geometry and coefficient of consolidation of the soil which controls the rate of decrease in effective stress and hence the swelling.

In materials of high permeability the coefficient of consolidation is high and the horizontal effective stress in the ground reduces rapidly to the net slurry pressure σ'_h' Figure 13. In sands and gravels the reduction of effective stress for the 'at rest' value to the net slurry pressure results in only very small swelling displacements and, provided the factor of safety against collapse is adequate, only small movements occur.

When diaphragm walls are constructed in materials with a high swell capacity the relationship between the construction time and the coefficient of consolidation is critical.

In sedimentary clays the coefficient of consolidation is normally low and during the time required to dig the panel, place the reinforcement cage and concrete, little swell occurs and ground movements are small.

Decomposed granite has a combination of a high coefficient consolidation with a high swelling capacity. The relationship between effective horizontal stress and swelling for samples of decomposed granite are shown in Figure 13. Because of this combination of high coefficient of consolidation and high swelling capacity the sides of the excavation move in during excavation and excess material is removed leaving behind soil which has swelled under an effective stress equal to the net slurry pressure in the affected zone behind the panel.

When the concrete is placed in the panel the horizontal total stress is increased and some immediate deformation may be recovered but, as the concrete sets, the effect of the concrete push is reduced. The net result of the panel construction is a loss of ground following expansion during digging, possible additional swelling before the concrete is placed and the creation of swelled, and hence compressible zone, adjacent to the panel in which the horizontal effective stress is only slightly greater than at a depth of 15m with an excess slurry head of 1.5m is about 25kN/m^2 and, as can be seen in Figure 13, considerable swelling can take place at these low effective stresses.

When adjacent panels are constructed the arching process throws large stresses on to the compressible zone formed behind the first panel and the soil in this zone consolidates. This causes horizontal movements further back from the wall which leads to ground settlement as illustrated in Figure 14.

In the final state the horizontal ground movement is that associated with panel the reconsolidation of a zone of about one half the panel width from the wall due to an increase in earth pressure from the net slurry pressure to the final in-situ earth pressure. This final pressure will depend on the total movement in the ground and will lie between the active and 'at rest' pressures.

The minimum value of the net slurry pressure to be considered during the excavation of individual panels is that which will cause collapse of the trench. To estimate the pressures exerted by the ground account must be taken of arching in the soil as this considerably influences the pressures acting on the sides of the trench.

The problem of arching around the trench is complex and involves the redistribution of stress in both the vertical and horizontal directions as movements of the soil occur.

The earth pressures at failure acting on the sides of the trench can then be estimated by conventional arching theory in a similar manner to that used in silo design if a number of simplify assumptions are made.

For the geometry of the excavation and groundwater conditions at the test panel, the earth pressures at failure are shown in Figure 15 for two assumptions regarding the shear strength of the soil. Line 'A', Figure 16, assumes zero cohesion in the soil whilst line 'B' includes a small effective cohesion of 7 kN/m^2 in the fill above the water table and in the decomposed granite at depth. From this diagram it may be seen that even a small amount of cohesion in the soil substantially reduces the earth pressures acting on sides of the trench.

In Figure 17, the calculated earth pressures at failure (Pź) are compared to the net slurry pressure supporting the trench (σ_h, Figure 12) in the test panel excavation. In the test, an increase in movement in the fill and marine deposits was observed when the slurry level was reduced to +2.6m P.D. (0.4m above the water table) and collapse of the upper section of the trench finally occurred with the slurry level at +1.63m P.D. (0.57m below the water table). By comparing the net slurry support provided at these stages of lowering the slurry and the calculated earth pressure, it may be seen that the observed behaviour can be accounted for using the model described and small amount of effective cohesion in the soil.

The model and analysis described for estimating earth pressures is probably an oversimplification of the real problem. However, it illustrates that arching in the ground can permit the horizontal effective stress during the construction of individual panels to be reduced to remarkably low levels without collapse of the trench occurring and, in decomposed granite, it is at these low stress levels that much of the swelling occurs.

The test panel excavation shows that considerable ground movements can occur in decomposed granite due to swelling well before the stability of the trench is at risk. As additional panels are constructed, movements extend back from the wall and, in many situations, it is the control of these ground movements that is of concern rather than trench stability.

As the prime cause of the movement is the swelling of the soil which takes place during the excavation of the individual diaphragm wall panels the only way to control the movement is to limit the swelling during excavation.

As has been shown in Figure 13 the swelling of the decomposed granite is highly non-linear and a marked increase in swelling occurs at low effective stress. This behaviour was also measured in the test panel and accounts for the observations made during the full scale installation where a reduction in net slurry pressure from an average of about 25 kN/m^2 at the Hong Kong Club to about 15 kN/m^2 at the Courts of Justice due to a rise in ground water level resulted in an increase in maximum settlement from around 30mm to 80mm. Thus, if the swelling pressure during excavation can be increased the swelling can be greatly reduced.

The swelling pressure is controlled by the net slurry pressure and this must be kept as high as is practicable. Where groundwater levels are high it may be necessary to lower the groundwater level during the construction of the diaphragm wall. Groundwater lowering will itself lead to settlements and the design must aim at obtaining the minimum combined settlements for both processes.

At one critical section of the diaphragm wall at Chater Station, ground movements beneath isolated piled foundations of a twenty two storey building 1m from the trench were controlled by maintaining the excess slurry head 7m above the groundwater pressures using well points and raising the slurry head above ground level using a high guide wall. In this case, horizontal movements in the decomposed granite during individual panel construction were limited to about 14mm and total building settlement to about 20mm.

The large settlements observed during diaphragm wall construction were associated with the fact that the decomposed granite has a high swelling capacity combined with a high permeability permitting rapid changes in effective stress to occur. This combination of soil properties is unusual which is why large settlements have not been observed elsewhere.

In sedimentary deposits, a natural sorting process occurs during deposition which tends to separate the clays from the more permeable silts and sands. However, decomposed granite is formed in-situ by chemical action resulting the production of clay minerals of high swelling capacity within a coarse permeable matrix and it is this difference in origin which accounts for the different behaviour.

3.0 Island Line

The Island Line is 14km long railway constructed almost entirely in tunnel and passes along the northern edge of Hong Kong Island. The running tunnels were mostly constructed adopting shield methods.

The Island Line was a contrast to the MIS system in that the stations were built off the line of the route adjacent to the main roads and connected to the running tunnels by cross tunnels. The consequences of this system of stations was that one of the tunnels needs to be quite deep and hence deeper concourses. As a result the tunnelling costs increased as the consequent higher pressure compressed air was required. This reduced the disruption on roads compared with the cut and cover techniques on the MIS. The Island Line proved to be very successful with only limited geotechnical problems despite the depths of the station excavation which were upto 27m deep. The line is 14km long and runs along the north side of Hong Kong Island.

The major input in the design therefore was to take the observations from the MIS system and apply to the Island Line is a cost effective manner.

An example of this is Wanchai MTR Station which is discussed below:

Wanchai Concourse

As part of the construction of the Mass Transit Railway Corporation's Island Line underground railways, Wanchai Station was constructed in the heart of urban Hong Kong (see Figure 2)

The station was constructed within a concourse "box" 60 metres by 40 metres in plan. The concourse box was formed by 1200mm thick concrete diaphragm wlls and the station constructed using the top down method to a depth of approximately 27 metres below ground level.

The site is located on the reclaimed foreshore on the northern edge of Hong Kong Island. The site was reclaimed between 1921 and 1932.

The geological profile at the site is typical of the northern edge of Hong Kong Island and similar to Chater Station.

Ground level (+4mPD) to -5mPD	Completely Decomposed Granite Fill
-5m to -10mPD	Sandy Marine Deposits and Alluvium
-10m to -35mPD	Completely Decomposed Granite
-35m to (-42m to -60m)	Moderately to Highly Decomposed Granite
below -42/-60m	Granite Bedrock

A geological section through the site is shown in Figure 18.

The groundwater level is approximately 2 metres below ground level. In the Fill and Marine Deposits the groundwater is effectively recharged by the nearly Victoria Harbour.

As discussed in the section above on Chater Station that in the decomposed granite soils of Hong Kong groundwater lowering associated with the excavation of deep basements results in significant ground movements.

Because of the history of ground movements during the construction of the first underground railway line in Hong Kong the designers were required to take all possible measures to minimise the effects of construction on the surrounding areas.

It was clear therefore that the design of the concourse box needed to take careful account of the effects on the groundwater regime.

The concourse required an excavation approximately 27 metres deep some 25 metres below the groundwater table. At an early stage in the design it was decided to use a top down method of construction within diaphragm walls around the perimeter of the site. In this way the stiff basement structure would minimise the movements due to excavation. It was essential that good dewatering of the concourse box was achieved to ensure the excavation within the box could be carried out effectively and quickly.

The construction sequence to be adopted is shown in Figure 19. Additional strutting was required for the lowest 2 dig levels to minimise ground movements and bending moements in the diaphragm wall.

It was clear as construction proceeded that the installation of the flying struts would limit the rate of construction and the significant cost savings could be achieved if the strutting could be reduced. By a reanalysing the movements that occurred for excavation to the 3rd basement it was possible to delete the strutting at the 4th basement level. This saved considerable costs and time on the station construction. The revised sequence is shown in Figure 20.

4.0 Airport Railway

The Airport Railway consists 34km of railway line linking the Hong Kong new airport at Chek Lap Kok and Lantau Island with Kowloon Hong Kong Island. The Airport Railway consists of 2 train services, the Airport Express and Lantau Line. The route is shown on Figure 21.

The railway is to be built for a significant proportion of its length either at grab or in tunnel constructed on recently reclaimed sand fill.

The major difference therefore with respect to geotechnical matters on the Airport Railway compared with the earlier MIS and Island Line projects is that the major requirement of any excavation support system for the tunnels is to maintain stability rather than to limit ground movements.

This brings a different approach to excavations. The need to adopt thick, deep, heavily reinforced and strutted diaphragm walls for the construction of stations and cut and cover tunnels is of less importance.

The approach will be to adopt either open cut excavation with a suitable ground water cut off where space is available, or use of sheet piles or diaphragm walls acting as cantilevers or with simple strutting.

As an alternative, temporary works excavation adopting sheet piles or thin precast cantilever diaphragm walls are being considered. In these situation movements would be large but provided stability is maintained this would not be critical.

Figures 22, 23 and 24 illustrate alternative options.

This precast diaphragm walls incorporating deeper slurry cut offs for groundwater control can provide a significant advantage if the wall is built for the permanent condition. The depth of the structural element would be determined by stability whilst the slurry cut off depth can be deeper to control seepage.

Some of the critical geotechnical aspects that are need to be addressed include.

4.1 Settlement of Sand Fill

The reclamation material under the major length of the railways will be pumped sand. In some instances the sand fill is to be vibro compacted prior to railway construction. For the major area at the south of Kowloon peninsula adjacent at Kowloon the treatment is specified. Beneath the sand fill are typically part clayey Alluvial Deposits and Completely Decomposed Granite. Although consolidation settlements in these deposits will be almost compete prior to construction, the sand, because of its loose condition, would be subject to additional settlement due to additional structural loads if placed on the fill, and also to vibration settlements from the operation of the railway and seismic events. The cost of vibro compaction of such large areas is very high and therefore designs are attempting to limit any treatment. Any treatment would be integrated in the studies on ground support systems for the excavations since densification of the fill could assist in providing more economical retaining structures.

4.2 Seismicity

Because of the loose condition of the sand fill, extreme seismic events could lead to liquefaction and settlements.

A comprehensive study of the risks of liquidation is being carried. Figure 25 shows typical return periods of seismic events against density of the sand fill and its susceptibility to liquefaction. These studies are being carried out for the Airport Railway to determine if additional treatment is required. It may be such that limited zones of the station are treated to reduce the risk of liquefaction.

4.3 Groundwater Level

Because of the relatively low loading of the station and tunnels flotation of the various structures will be a key element to investigate.

Underdrainage systems to reduce the hydrostatic water pressure are now extensively used in Hong Kong. It is probable however that such systems will have limited applications for the MTRC because the fail safe system with this method is to allow the structure to flood - this is clearly unacceptable on a railway. It may be possible to adopt the development underdrainage system in non-railway areas of the station or provide additional fail safe systems.

Small diameter mini-piles in tension are currently being used on provide where depths of excavations are similar to the major portion of the Stations.

References

DIBIAGIO, E. and MYRVOLL, F. (1972), "Full scale field tests of a slurry Trench Excavation in Soft Clay", Proceedings, 5th European Conference on Soil Mechanics and Foundation Engineering, Madrid, Vol. I, pp461-471.

HUDER, J. (1972), "Stability of Bentonite Slurry Trenches with some experience in Swiss Practice", Proceedings, 5th European Conference in Soil Mechanics and Foundation Engineering, Madrid, Vol. I, pp517-522.

KARLSRUD, K. (1975), "Practical Experience from the Excavation of Slurry Trench in Oslo Clay". Norwegian Geotechnical Institute Publication No.110.

LUMB, p (1979), "Building Foundations in Hong Kong", Proceedings 6th Asian Regional Conference on Soil Mechanics and Foundation Engineering, SIngapore, Vol.2, pp211-214.

MACKEY, S and YAMASHITA, T. (1967), "Experience with Caisson and Pier Foundations in Hong Kong", Proceedings, 1st S.E.Asian Regional Conference on Soil Mechanics and Foundation Engineering, Bangkok, pp301-312.

MEYERHOF, G.G. (1972), "Stability of Slurry Trench Cuts in Saturated Clay", Proceedings, Specialty Conference on Performance of Earth and Earth Supported Structures, Purdue, Indiana, A.S.C.E. Vol. I, Part 2, pp1451-1466.

DAVIES, J.A. (1087), "Ground Water Control in Design and Construction of a Deep Excavation", Proceedings of the Ninth European Conference on Soil Mechanics and Foundation Engineering, Dublin/31 August - 3 September 1987

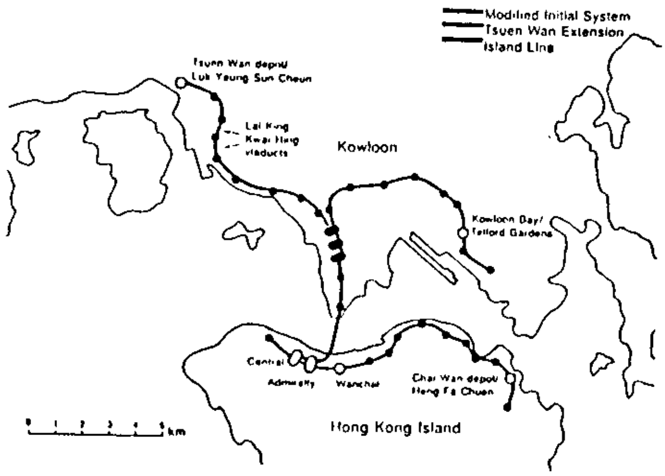

Figure 1
MTRC

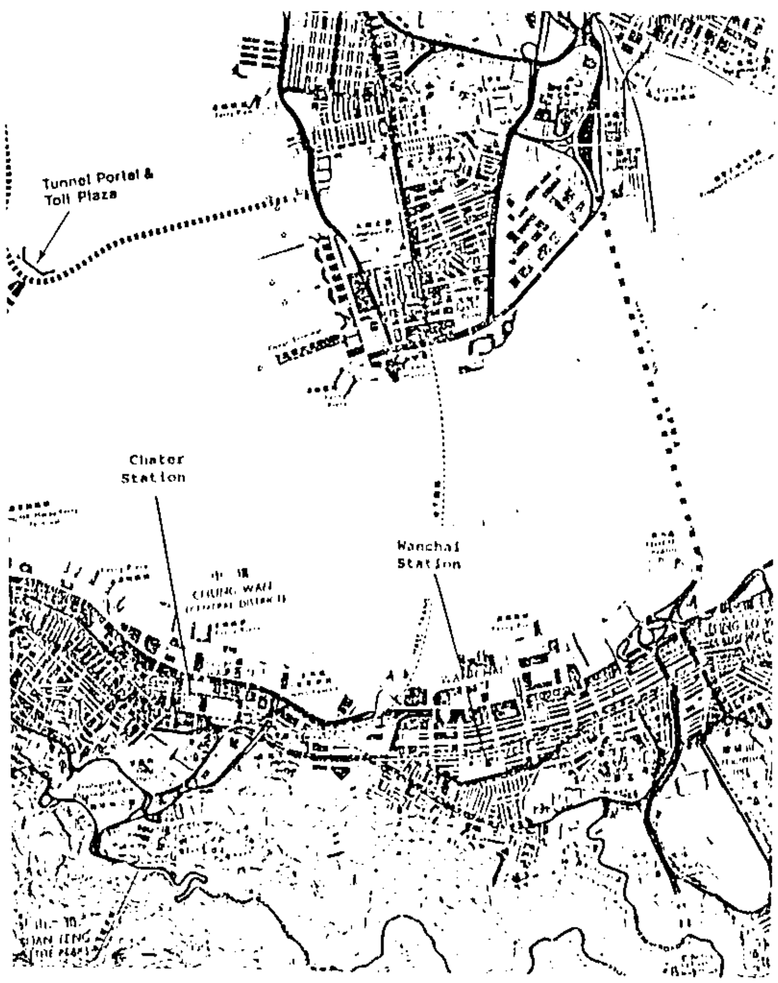

Figure 2
SITE LOCATION

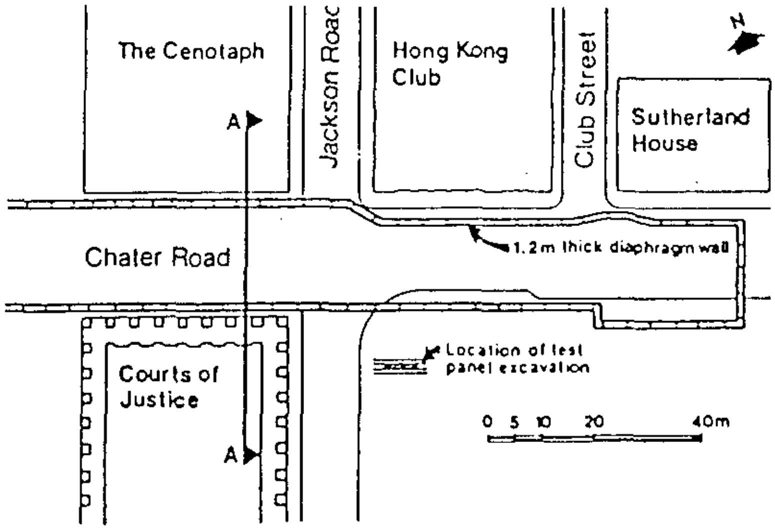

Figure 3
PLAN OF DIAPHRAGM WALL

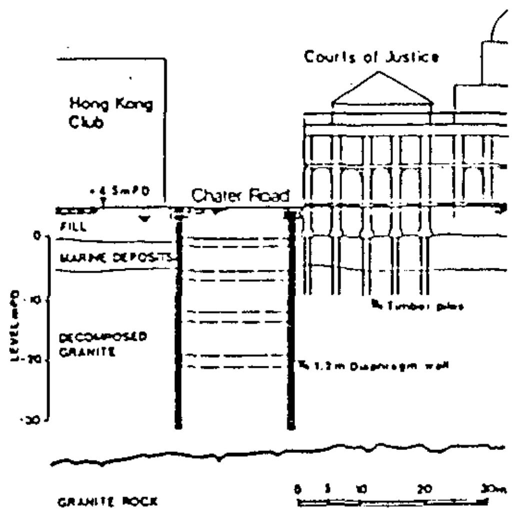

Figure 4
SECTION AA

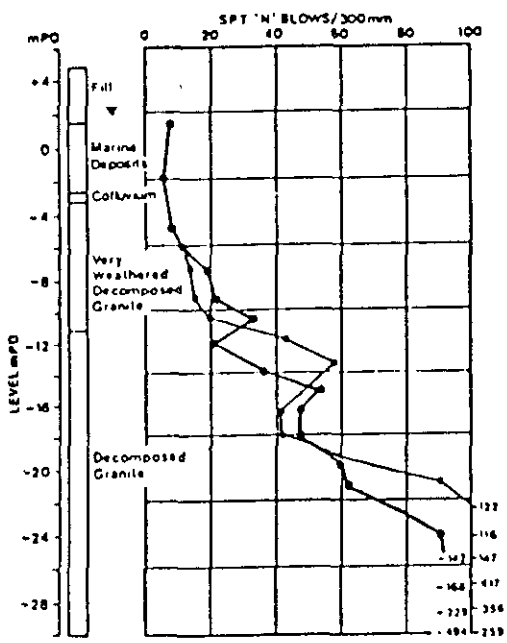

Figure 5
SOIL PROFILE

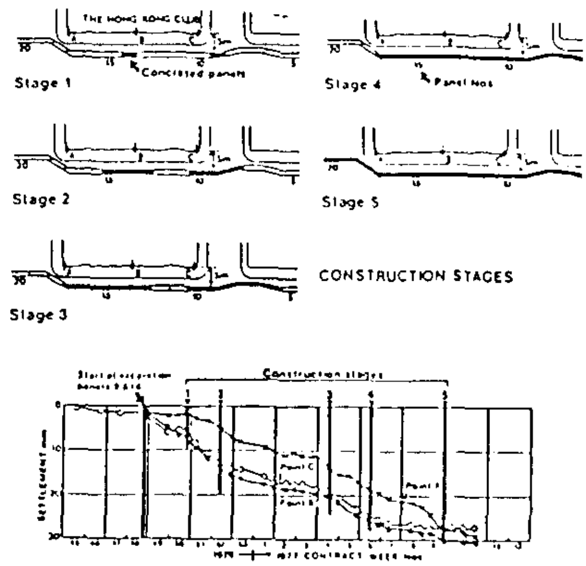

Figure 6
SETTLEMENT AT HONG KONG CLUB

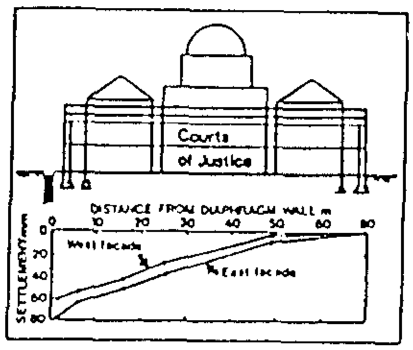

Figure 7
SETTLEMENT AT COURTS OF JUSTICE

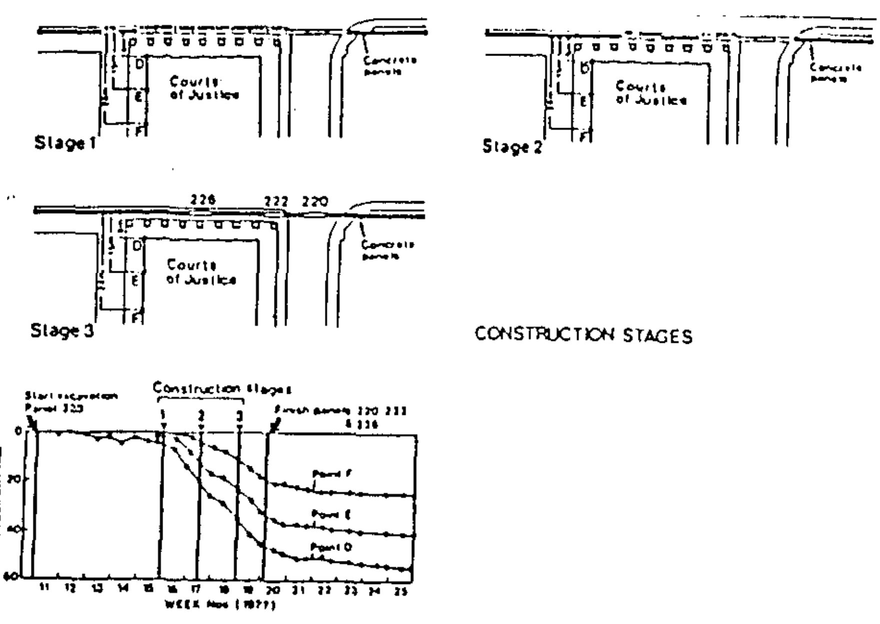

Figure 8
DEVELOPMENT AT THE COURTS OF JUSTICE

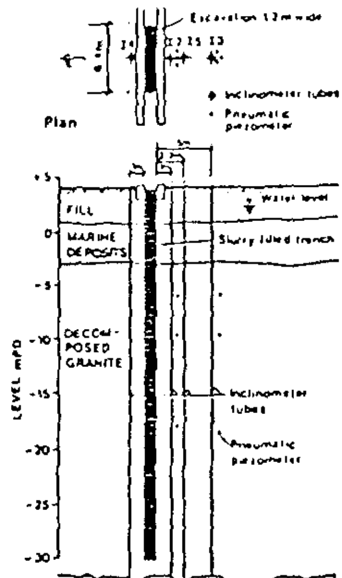

Figure 9
TEST PANEL EXCAVATION

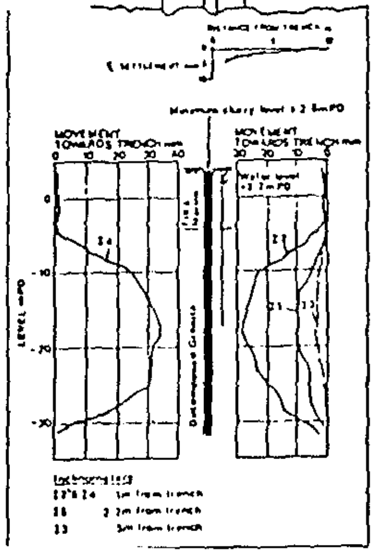

Figure 10
HORIZONTAL MOVEMENTS

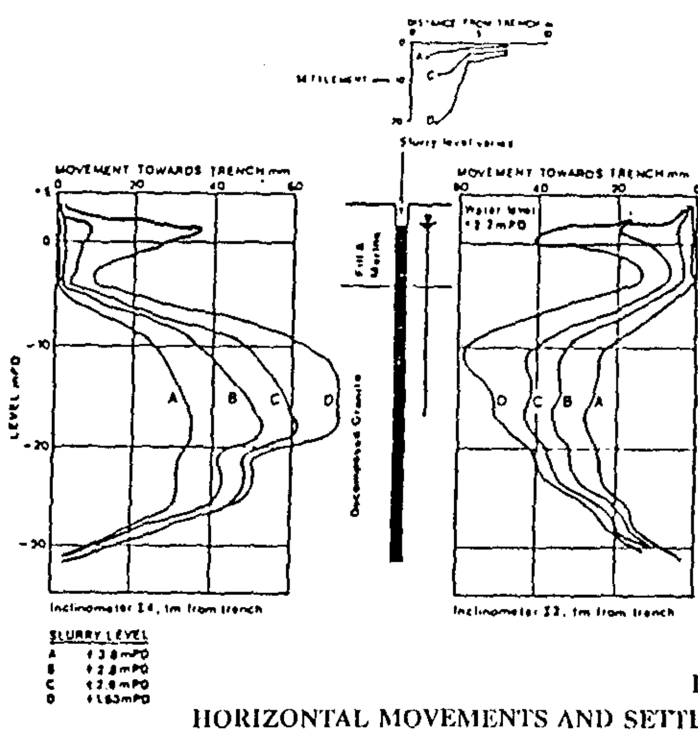

Figure 11
HORIZONTAL MOVEMENTS AND SETTLEMENT
DUE TO LOWERING SLURRY LEVEL

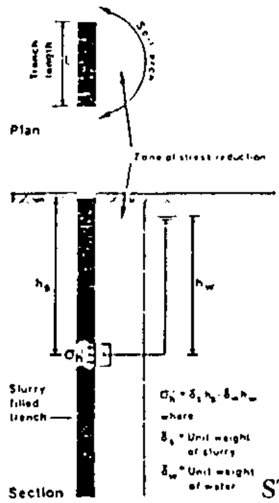

Figure 12
STRESS CHANGE ADJACENT TO PANEL

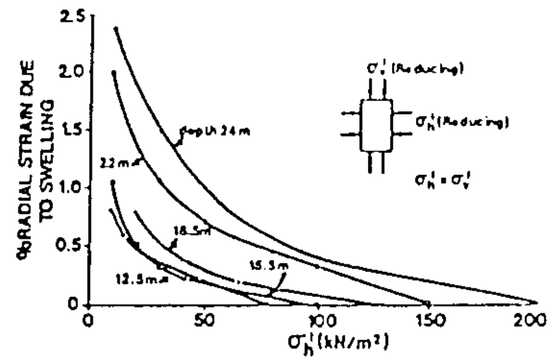

Figure 13
TRIAXIAL SWELLING TESTS

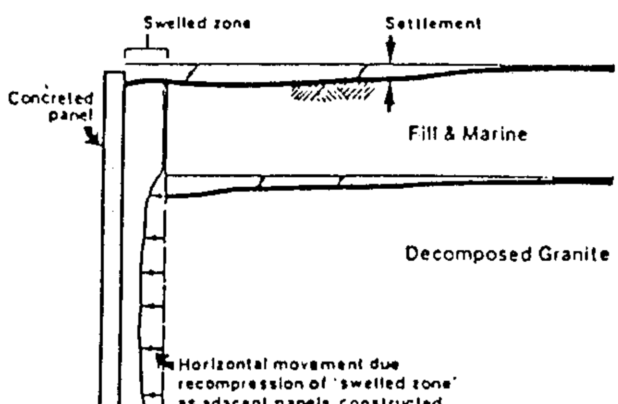

Figure 14
GROUNDMOVEMENTS DUE TO CONSTRUCTION
OF DIAPHRAGM WALL

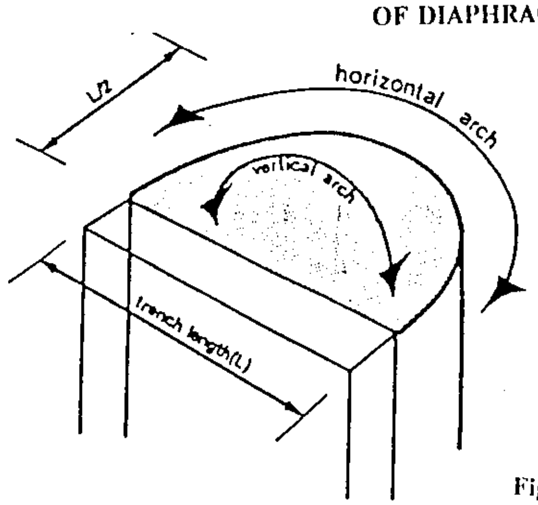

Figure 15
SOIL ARCH

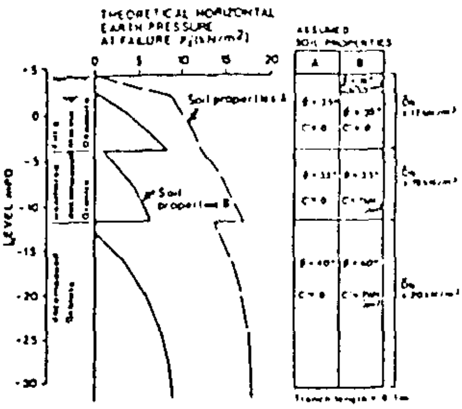

Figure 16
EARTH PRESSURE AT FAILURE

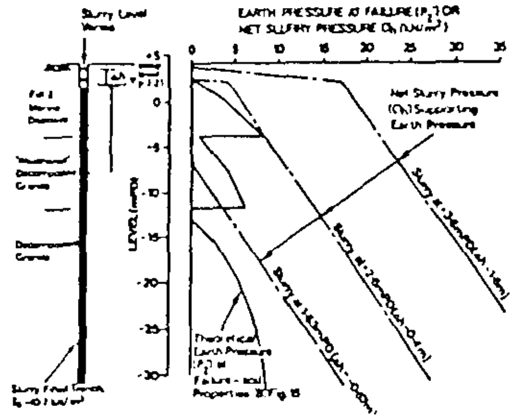

Figure 17
EARTH PRESSURE AND NET SLURRY PRESSURE

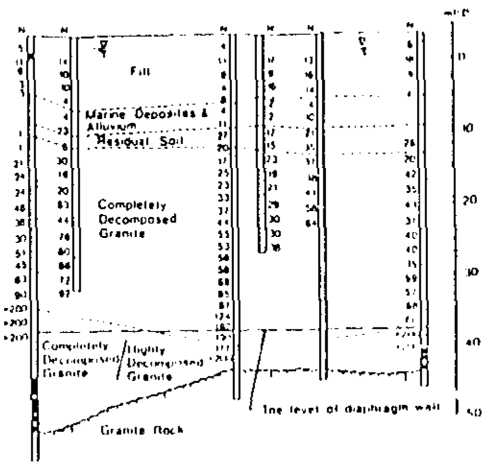

Figure 18
SOIL PROFILE

Figure 21
AIRPORT RAILWAY ROUTE

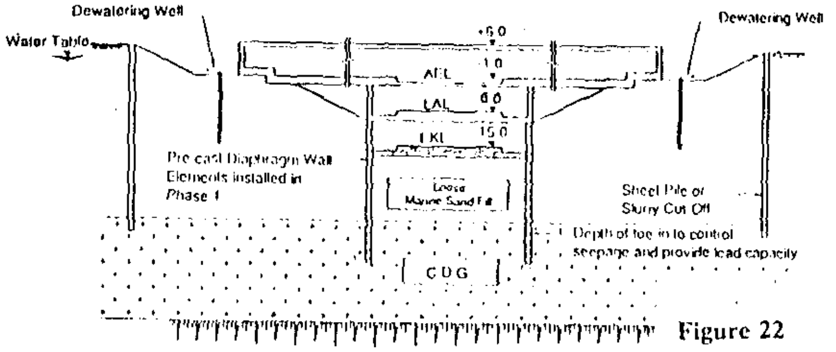

Figure 22

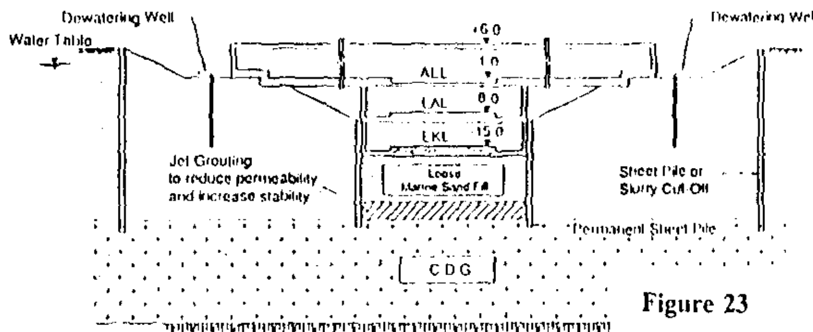

Figure 23

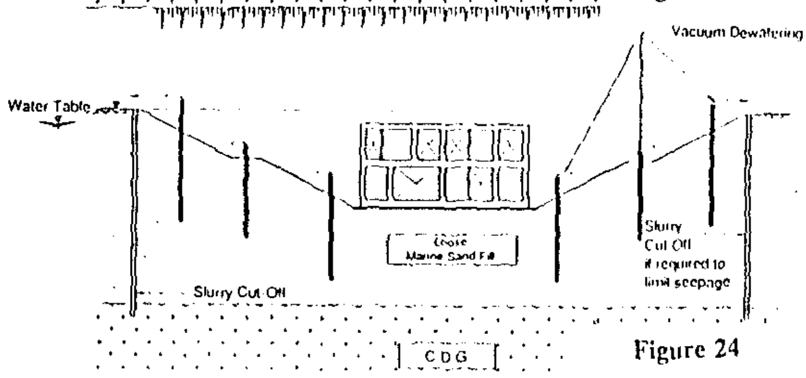

Figure 24

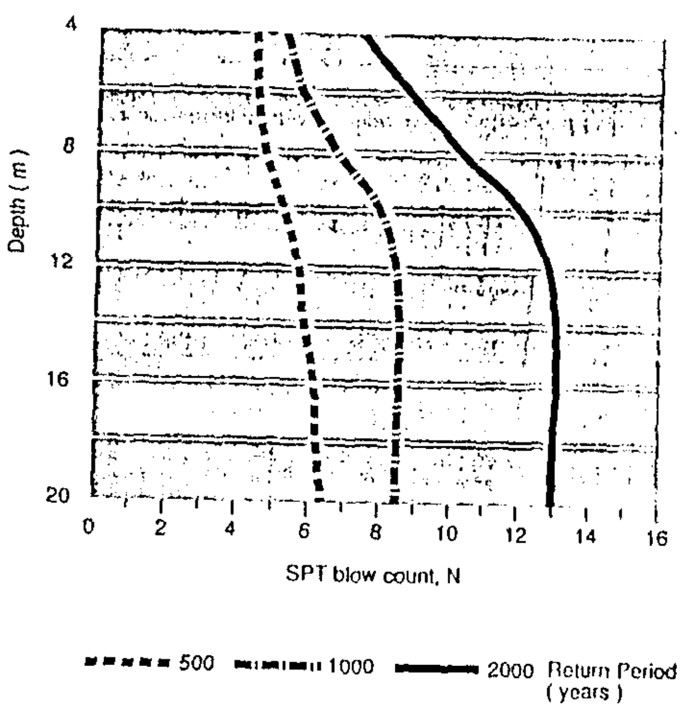

Figure 25
LIQUIDATION POTENTIAL