7th Lumb Lecture 10th October 2012 "Peter Lumb's Legacy, Soil Mechanics = Simple Concepts +Mathematical Processes + Lateral Thinking"

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SYNOPSIS: Professor Peter Lumb's legacy to the Hong Kong geotechnical engineering profession was 32 years of service at the University of Hong Kong. For this he is fondly remembered by his many students as a quiet teacher, a contemplative man. The majority of his time Peter had grappled with tropical weathering and its consequence in engineering properties as well as the performance of soils and rock in an industry that was mostly not very enlightened for some 24 years before the Geotechnical Control Office (GCO), was established.
In his early days reliable laboratory testing was not common. Peter built the first testing laboratory in Hong Kong . Computers were under development and not in use. Peter taught assessment, insight and auditable hand calculations. Faced with a heavily regulatory system designed to compensate for inadequacies of the not well informed amongst the practitioners, he shied away from getting involved with day to day projects. As a profound thinker, when Ken Roscoe at Cambridge University was working on Critical State Soil Mechanics and Alan Bishop at Imperial College London was trying to perfect uni-axial compression tests, Peter realised that statistics was a means of handling variation, uncertainty and risk. Like some other geotechnical people, trained to investigate, he branched out into a new field and became a worldwide specialist in statistical theory not related to applications to soil mechanics.
He retired 26 years ago. What have been the fruits of his legacy? The most obvious results are dozens of his former students who have carried on his tradition, not necessarily in soil mechanics, and have achieved high positions and led worthwhile lives. The industry has changed. Testing laboratories are accredited. Deep excavations with lateral support and foundations are designed rationally. Much reclamation have been completed without the mud waves of the kind that were generated in the 1970's. Thanks to the efforts of the Geotechnical engineering Office (GEO), landslide risk has been significantly reduced. The subject of stability of slopes is complex and there is fascinating on-going research into the performance of slopes. Computers are taken for granted. Computations can be carried out quickly and more intricately than he imagined. Mathematics was a predictive tool, now it is hidden behind icons which can be invoked without thought.
Mathematics has been a principal tool behind the soil mechanics that Peter taught. Coulomb and Terzaghi were mathematicians. However solutions have given place to processes. Numerical modelling is very useful and is now made freely available to engineers. The collapse of the Nicholl Highway in Singapore was initially blamed on the mis-use of numerical modelling. Within limits debris flow can be analysed but prediction of flow remains difficult. Numerical models can predict slopes moving uphill in the dry season.
Statistics are being adopted to a limited degree. Quantitative Risk Assessment and Fractal Analysis require large supplies of relevant data. Today gigabites of data are transmitted in minutes. One wonders whether Peter would have approached statistics in a less theoretical way had he been working 26 years later?
Geotechnical Engineers file data spatially as Geographic Information Systems (GIS). Very much as Peter thought laterally and was attracted to statistics likewise GIS people, thinking laterally, have moved into asset management and a variety of other fields.
The legacy of Peter Lumb lives on; it is the better side of human nature.

Professor Peter Lumb

1. INTRODUCTION

The Lumb Lecture is held in Hong Kong biennially to celebrate the work and the legacy of a great Geotechnical Engineer, Professor Peter Lumb. Peter joined the Department of Civil Engineering of the University of Hong Kong in 1954 and retired as Professor of Civil Engineering in 1986. He pioneered local research on the behaviour of tropically weathered soils and slope failure along with many other aspects of geotechnical engineering. He was a dedicated teacher and an international leader in the area of geotechnical reliability.

Peter Lumb retired in 1986, that was 26 years ago. Even his youngest students are nearing retirement. I had the privilege to meet Peter in 1975 and was able to get to know him for 11 valuable years before his retirement. Therefore I can only reminisce from 1975 onwards.

His work is well reflected in his paper and writings of which some 65 have been edited in a volume published in 2002, (Yeung 2002). He wrote about a wide range of subjects including consolidation, settlement of buildings, foundations, piles, soils of Hong Kong, rainfall, slope stability, soft marine clay, probability of failure, spatial variability of soil properties, land reclamation, progressive failure, and probabilistic design of slopes and so on. From 1966 to 1971 he supervised nine M.Sc. students and from 1972 to 1981 there were ten M.Phil. students.

This paper reflects on what he taught the geotechnical engineering profession in the context of the state of practice at the time and his legacy, that is to say what has been learned from his teaching.

2. PRIOR TO HIS RETIREMENT IN 1986

Peter was a teacher and a fundamentalist. He recognised that all geotechnical work is based on site characterization and that above all else one needs to start with understanding the geology, the condition of the ground, specific to the site. In those days the fundamentals of soil behaviour were barely studied, tools to handle complex constitutive relationships were weak and empiricism was the most reliable approach. Correlation of performance was derived from data from the field. Hand calculations were advocated. They were straightforward to understand and in 1975 few offices had computers and they were not widely used in 1986.

Peter was very well respected internationally as a geotechnical practitioner, but there was a shortage of geotechnical engineering capability in the industry in Hong Kong. He certainly was a pioneer. One could say that he was a prophet in his time!

Peter did not give up on the profession. Instead he worked at it. In his early days he researched and taught extensively. Peter wrote many good papers on diverse geotechnical subjects as mentioned above.

One of Peter's contributions to the geotechnical engineering profession in Hong Kong was the Wetting Band Concept. The conceptual model was simple. When rain falls on a slope, water seeps into the soil and descends to the phreatic surface resulting in a rise in the ground water level. The concept is now well used in Hong Kong. Many slopes have been designed with a rise in groundwater level derived according to Peter's wetting band equations.

The Wetting band assumes flow of water through the soil under gravity and therefore the hydraulic gradient is 1.0. The calculation adopts the value for permeability to water of saturated soil. The more astute reader will note that the infiltration from above the phreatic surface is where the soil is partially saturated and its permeability to water is much less than in saturated soil. Was he right?

The wetting band model can be put to the test. Figure 1 shows three tensiometers inserted into a natural slope to depths of 1m, 2m, and 3m respectively. Tensiometers can measure positive ground water pressure and suction of up to one atmosphere above which cavitation occurs in the tensiometer. For example, readings of the tensiometers were taken on 16th September 2005 and 23rd September 2005 during which period there was virtually no rain.

Figure 1 Tensiometers at 1m, 2m, 3m depth

There was 275mm of rain on 25th September and further readings were taken on 27th September 2005 and 30th September 2005. The measurements are shown plotted against depth in Figure 2. These show that on the dry days from 16th to 23rd September the suction increased. On 27th September, the first readings were taken after rain by which time the suction had dropped and was almost zero at 1m depth but was still over 40 kPa at 2m depth. However by 30th September the suction had dropped to almost zero at 2m depth. By reference to the profile for 27th September on Figure 2, with no suction at 1m depth and over 40 kPa suction at 2m depth there was a pressure gradient of over 50kPa/m, more than 5 times the gradient due to gravitational flow.

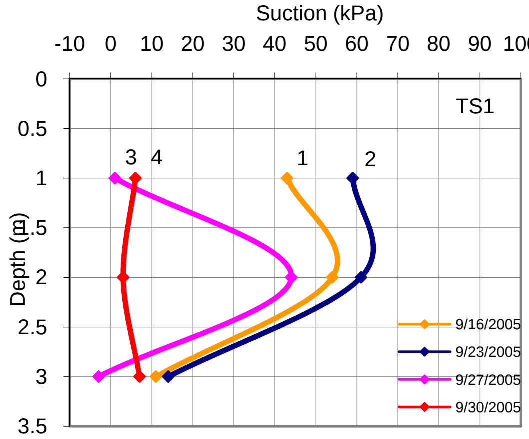

Figure 2 Profiles of suction before and after rainstorm

Peter's simple model assumed permeability of saturated soil whereas the permeability is lower when soil is partially saturated. However his model assumes gravitational flow whereas above the phreatic surface the pressure gradient can be five times that due to gravity according to this example of field data. With these two effects offsetting each other, Peter's approximation appears to work. Nowadays sophisticated computer software, not available in Peter's time, can cope with such changes in suction, positive pore water pressures, permeability and changes in hydraulic gradient. Nonetheless, simple calculations are better understood than complex computer outputs.

2. WHAT DID PETER DO IN ADDITION TO TEACHING

Although Peter did not engage in a lot of consulting work, he was appointed to prepare reports on major events. These included regional subsidence of buildings in Mong Kok, Po Shan Road landslide and mud waves in the reclamation at Sha Tin. He engaged in research and published many technical papers, but he generally kept away from private building work. Having considered the frequency of slope failure and the spatial variation of soil property he turned his mind to probabilistic theory and found a new interest in pure statistics.

3. THE STATE OF PRACTICE DURING HIS TIME

4.1 Slope failures during rainstorms

In the1970's it was accepted that roadside cuttings often slipped during rainstorms. After all, a gang of labourers with shovels could throw the soil over the edge and the road would be cleared for traffic shortly afterwards. However from time to time rainstorms resulted in disaster and complete sections of roads were lost, see Figure 3. More importantly some major slope failure resulted in fatalities as large volumes of debris were involved, see Figure 4. In 1976 a well-documented failure occurred at Po Shan Road which knocked down an apartment block and damaged another resulting in many fatalities, see Figure 5.

It is not surprising that Peter spent a lot of time studying slope stability. However in the 1970's there was a lot of geotechnical work to do and very few geotechnical engineers to do it! Peter's response was to focus on teaching, and train the team, rather than practice himself.

Figure 3 Collapsed road due to landslip

Figure 4 Large volume of debris from a major slope failure

Figure 5 Failure at Po Shan Road which demolished a 12 storey apartment block

4.2 Reclamation for new towns

An engineering feature of the 1970's was reclamation, forming new land in marine areas for new towns such as Sha Tin, Tai Po, Tuen Mun. The idea was to borrow soil from the hills and tip it as fill into the sea, see Figure 6. The formed platforms in the hills and the reclamation provided much needed land for development. Muck shifting is not high technology and in some case engineering input was not sought. For example, mistakenly, mud dredged from the trench for the sea wall was dumped inside the reclamation and, by end tipping of spoil from trucks the soft sea bed was displaced. Covering displaced mud with fill was a nightmare and the ground was not stable, see Figure 7. In some reclamation settlements of as much as 8m were recorded (Halliday 1982). At places mud came up to the surface, having risen more than 10m as a mud wave, see Figure 8.

Figure 6 Reclamation for Sha Tin New Town 1976

Figure 7 Unstable ground where mud had been displaced

Figure 8 Mud displaced by dumping fill

Peter Lumb studied consolidation characteristics and strength properties of soft marine mud. He published the work and discussed its implications on design of reclamations. Yet, as far as I know, he did not design reclamations; but he was called in to put right reclamations that had already gone wrong.

However, not all was wrong with reclamations. Tests were carried out to explore the use of vertical drains to consolidate the mud, including a large trial embankment at Chek Lap Kok in 1982 for which monitoring was sustained for nearly 16 years.

4.3 Basements

In the 1970s few buildings had any basements at all. High prices of rented space in Central created a need for several floors of basements in re-developments. Architects produced plans and contractors dug deep holes. However all was not well with design of earth lateral supports, see Figure 9 for example.

Figure 9 Collapse alongside excavation for a basement in Central in 1981

Designs for retaining structures were rudimentary. In one example there was a steel sheet pile wall with one row of inclined struts and no toe in, see Figure 10. Moreover a wide waling was used with an inclined strut and no holding down feature, see Figure 11. Who was to blame for the state of the steel work? Was it the Designer or the Checking Authority or the Contractor? It is therefore of no surprise that Peter held the local profession with disdain in those days.

4.4 New standards

All was not doom and gloom. In 1975 at the commencement of construction of the Mass Transit Railway, a bold decision was taken to tender the works for the underground railway by design and construct including some sections for local contractors. As I recall, at the time there was only one completed diaphragm wall in Hong Kong and there was no prior construction of underground railways in the region.

The MTRC established new standards of quality of supervision, quality of construction, and of project management. The phrase "on time and within budget" became popular. The local contractors offered skills and experience that were cost effective including new applications of traditional local methods for construction, see Figure 12. One of the successful local contractors, Paul Y Construction Ltd, had a close relationship with sub-contractors who excavated shafts by hand, called "hand dug caissons", see Figure 13. These proved to be less expensive than using slurry filled trenched for conventional diaphragm walls and caisson walls were invented. It is unfortunate that the workmen mostly refused to wear air filters and many perished from the dust when drilling rock and pneumoconiosis and the method of construction has been virtually banned. However it was a versatile method of constructing foundations and walls, see Figure 14.

Figure 10 One row of inclined struts and no toe-in for the steel sheet piling

Figure 11 No holding down of waling with inclined strut

Figure 12 Construction of the north wall for Diamond Hill Station

Figure 13 Excavation of a Hand Dug Caisson

Figure 14 Underground station walls constructed by hand dug caissons

4.5 Design of strutted deep walls

Several of the designs for the first excavations for the MTR made use of Terzaghi & Peck's trapezoidal envelope of soil pressures, see Figure 15. These envelopes were derived empirically and were used to estimate loads in struts directly. They were hand calculations.

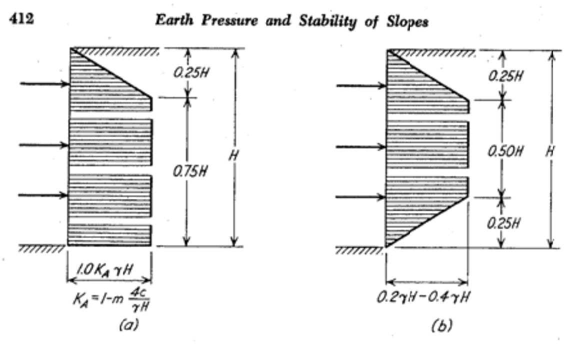

Figure 15 Terzaghi & Peck's pressure envelopes

However the envelopes were for clay soils and were not necessarily completely weathered and residual soils. Also, as general guidelines they were not site specific and were likely to be conservative. At the time numerical modeling was not available commercially. Soil/structure interaction was also not in use for design but its application offered economies. Early designs in 1975 utilising soil/structure interaction whereby as the walls moved inwards the external soil pressure dropped from the at-rest pressure to the active pressure and as excavation proceeded the passive pressure developed inside the walls. The hand calculations were tedious and simple "beam on springs" computer programs were soon developed.

In 1977 the Geotechnical Control Office was established in Hong Kong firstly to address the stability of slopes and later became involved with ground engineering more generally. By 1982, Geotechnical Control Office issued Geoguide 1 for design of retaining walls (GCO 1982).

4. WHAT HAS HAPPENED IN THE PROFESSION AFTER PETER LUMB'S RETIREMENT

There have been many and far-reaching changes in the 26 years since Peter retired. A major change has been the replacement of programmable calculators (PCs) by personal computers (PCs). The advent of computers opened opportunities to save a lot of hand calculations. Moreover, in his time there was a shortage of data now electronic data files can transmit and sort gigabytes of data. Extensive monitoring provides lots of data that creates new opportunities for empirical correlation. Peter practiced empirical correlation but he did not have access to the large amounts of data that are now available.

5.1 Numerical modeling

An early example on the use of numerical models is a private building on a steep site, see Figure 16. The overall slope was about 50 metres high at a slope of 1:1. It was thought to be a soil slope and potentially dangerous. The building site is located at the top portion of the slope above the road. Stabilising work had been carried out on the lower portion of the slope where horizontal drains had been installed, see Figure 17, (Tong and Maher 1975). The flow of water from the drains had been monitored and the flow was concentrated amongst only a portion of the drains, as shown plotted in Figure 18.

Figure 16 A building site on a seep slope

Figure 17 Installation of horizontal drains in steep slope

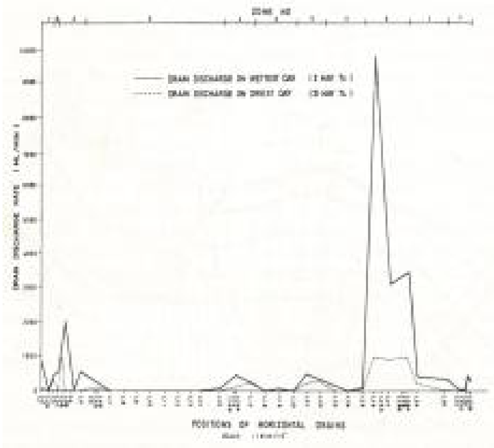

Figure 18 Flow of water from horizontal drains (After Tong and Maher 1975)

Many site investigation borings were carried out and a 3-D model was made from styrofoam and brass rods painted to show rock, saprolite, and residual soil, see Figure 19. It is evident that most of the ground was rock, shown black in the figure, with distinct, deep weathering, shown dark grey. The original interpretation that the slope comprised entirely soil was based on few boreholes located in zones of deep weathering. The zones where the rock levels were high had not been recognised.

Figure 19 3-D geological model of site on hillside

Development of such a steep site required retaining structures and the design included two rows of hand dug caisson walls, interacting, one above the other. The passive zone of the upper wall impinged on the active zone of the lower wall. This project was the first use of a computer program with a numerical model treating soil as a continuum. One cross section is shown in Figure 20. The design was approved in 1986 in the year when Peter retired. Figure 21 shows the site during construction. Project was completed and occupied for many years but it was later re-developed with an even taller apartment block.

During the last 26 years computers have become much more powerful and programs are far more versatile. Several applications of more advanced computing are described below.

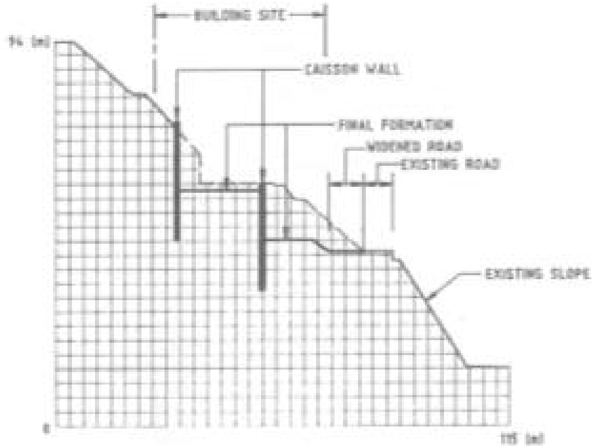

Figure 20 2-D numerical model of double walls

Figure 21 Site formation work in progress on steep site

5.2 Lots of data - depth of weathering

With foresight, GEO established a Geotechnical Information Unit (GIU) which now holds archived records of about 300,000 ground investigation boreholes, as shown in Figure 22.

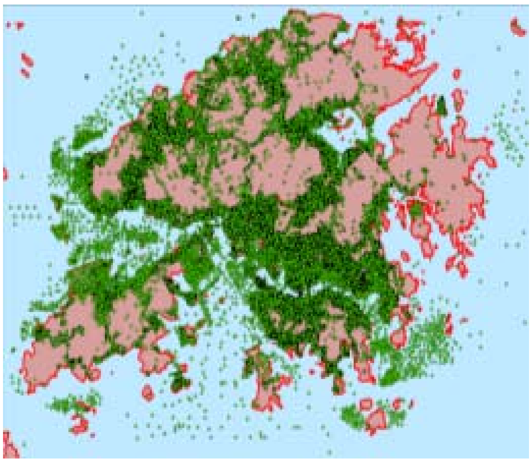

Figure 22 Locations of over 300,000 Archived Borehole Logs

Most of these archived boreholes report depths to rock. Early in the investigation of a site archived boreholes can be accessed. A commonly experienced task is to address the variations in levels of rock head based on data from a number of boreholes. In particular, what variations in level of rock head can be expected between the boreholes?

Given many boreholes in one location, a simple probabilistic approach is feasible. If, for every pair of boreholes the difference in elevation of rock is plotted against the distance between the boreholes, one can end up with a lot of points, see Figure 23.

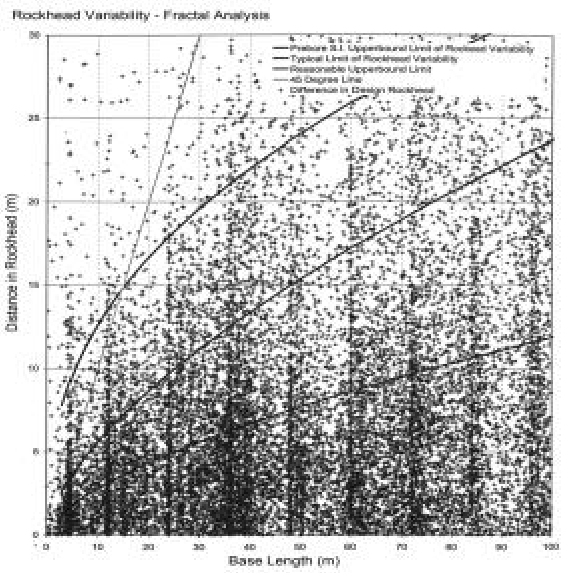

Figure 23 Difference in elevation of rock versus spacing of boreholes

As the spacing between the boreholes gets closer this plot shows the variations in the deeply weathered rock surface including some boreholes reaching corestones above bedrock and some boreholes penetrating deeply inside sub-vertical weathered joints. It can be seen that the variation is up to 30m difference in elevation and some points were above the top of the graph. It may be noted that the variations are similar even when the boreholes are closer than 10m spacing. From this data the depth of weathering is therefore of the order of +/- 30m at this location. Other locations have a different range. For example, the same assessment conducted for a site in Singapore shows a smaller variation, see Figure 24. For boreholes less than 20m apart the variation in elevation is less than 10m, less than a third of the variation at the site in Hong Kong as described above.

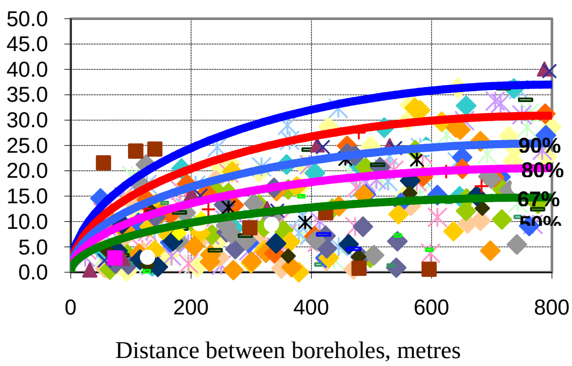

Figure 24 Variation in elevation of rock versus spacing of boreholes

5.3 Lots of data - conductive joints below rock head

Years ago, the amount of water that would flow into an unlined tunnel was a commercial secret for contractors who could use the information to their advantage when tendering tunnelling contracts. Those who lacked such knowledge and experience could lose a lot of money when tunnel contracts were awarded on the basis of the lowest tendered sum. Given packer tests in boreholes reporting flow of water ranging from 1 to 1,000 litres per minute and sparse geological data, it was not realistic to estimate the total flow of water into a tunnel within two orders of magnitude. Estimating the amount of grout needed to stem the inflow was guesswork in most cases. Employers considered re-measurement of the amount of grout to be a license for the Contractor to print money.

From 1996 until 2000 HATS Stage 1 sewer tunnels were driven at about 90m to 140m below sea level. Ingress of ground water during construction was a potential problem with regards to draw down of ground water pressures and consequential consolidation settlement of the overlying soils. For these tunnels, tunnel performance data was recorded electronically in a Tunnel Data Management System (TDMS).

Stage 1 TDMS recorded a lot of data including:

(a) Geological logs
(b) Number, and lengths of probe holes at each location
(c) Inflow of water from each probe hole
(d) Amount and type of grout used for each probe hole

The issue then was how to use such data to estimate flow of water into a tunnel below sea level?

As Peter would have done, one would look for a correlation between grouting and geology. Given that this was the first time for such a correlation and the data was limited to the six tunnels, possible discriminating factors were selected as follows:

(a)	Lithology :	Granite	or	Volcanics
(b)	Location:	Land	or	Marine
(c)	Alteration:	Fault	or	Not in a faulted zone (>25m/50m away)

There was data primarily from two rock types, namely from Granite of the Kowloon Pluton and Volcanics of the Repulse Bay Group. The location factor was in effect an division between generally a minimum cover of rock, aimed at 30m cover for tunnels beneath the sea and a much larger cover of rock, of the order of 100m to 300m for tunnels beneath land. It was thought that the amount of rock cover would relate inversely to the probability that conductive joints in the rock would interconnect and reach a supply of water. It was recognised that the conductivity of these igneous rocks tends to be affected by the local faulting and the degree of alteration. A criterion of close to a fault, or distant from a fault was determined by the degree of fracturing and turned out to be in the range of 25m to 50m. The TDMS data was classified according to these three parameters. However there was very little tunneling in Volcanics under land so there is data for six classifications of ground conditions and not for all eight.

In order to control subsidence due to dewatering, the total amount of water (per metre length of tunnel) has to be limited by flow rate and duration. For such tunnels a residual flow rate such as 1 l/min/m to 10 l/min/m might apply. In order to achieve such a flow rate any probe holes with a higher rate of inflow would have to be grouted. Therefore the rate of flow of water for each probe hole was sorted according to the six types of rock conditions. For each type of rock conditions the percentage of probe holes that had a flow rate less than the specified value was determined. The results are plotted in Figure 25.

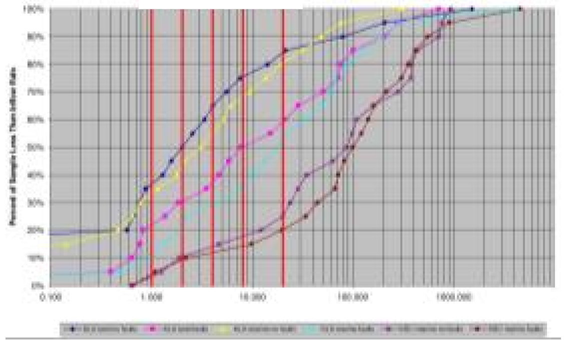

Figure 25 Percentage of probe holes with inflow rates less than the specified values

If one can determine the rock conditions along the length of a tunnel, one can correlate the percentage of probe holes that would exceed a set rate of inflow and therefore would need grouting. Given a geological model from which to derive the rock conditions along the alignment one can compute an estimate of the number of probe holes that would have to be grouted in order to stem the inflow down to the specified rate of inflow.

Likewise using TDMS data one can estimate of the amount of grout to be used. Such estimates for the number of probe holes to b grouted and the amount of grout to be used are of more use than trying to estimate the total inflow that might have occurred had the tunnel not been grouted. Such correlations depend on similarity of ground conditions under which the data was collected and ground conditions that are anticipated for the new tunnels. Such an estimate also depends on the reliability of the geological model.

Figure 26 shows part of a geological model including simplified logs for the Stage 1 tunnels as well as the mapped solid geology, lots of borehole data and other geological information.

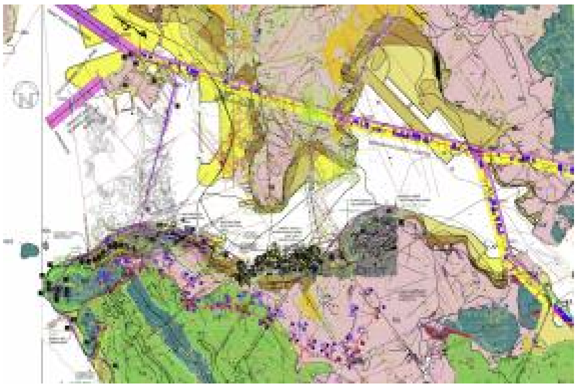

Figure 26 Computer based geological model of the Harbour Area

For every 20 m length of the tunnel, one needs to know:
(a) Granite or Volcanics
(b) Marine or Land
(c) Degree of Alteration

The key question is how can one determine the rock type for every 20m given vertical boreholes spaced more than 20m, say 200m, apart, and how can a Tenderer do it within a few weeks?

An answer is to make available as much geological information as possible in a digestible form. One method is to provide lots of cores of rock taken in the immediate vicinity of the proposed tunnel and ideally extensively along the route of the tunnel. Whereas vertical borings down to the level of the tunnel provides cores at one location, horizontal coring at the level of the tunnel can recover lots of core over long lengths of the tunnel. In Hong Kong horizontal coring has extended for more than four kilometers. Directional drilling has been adopted for ground investigation. Once the borehole is in rock it is possible to accurately steer the boring both horizontally and vertically. Therefore it is possible to use a site from which to drill that is off-line. The hole is drilled steeply down through the soil and once it is in rock it is steered onto the alignment and close to the tunnel, see Figure 27.

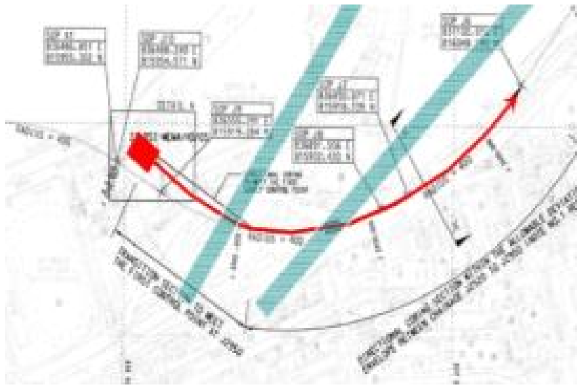

Figure 27 Directional coring to reach the alignment for a tunnel

Horizontal Directional Coring (HDC) has been used extensively for the HATS Stage 2a deep sewer tunnels in Hong Kong. Some 19.5kms of coring for about 26% of the length of the tunnels has been retrieved, including long sections of solid core and almnost total recovery in highly fractured rock, see Figures 28 and 29.

Figure 28 Long lengths of solid core recovery of Granite rock

Figure 29 Almost total recovery of highly fractured volcanic rock

Such long lengths of core provide a lot more information about the rock at the vicinity of the tunnels than vertical boreholes spaced at many metres.

The results from HDC can be incorporated into a geological model, as shown in Figure 26. A detail, part of a vertical section taken from the model at the location of one of the shafts is shown in Figure 30. The HDC core is shown as a simplified geological section across the bottom of the figure.

From a geological model such as this it is a relatively straight forward task to classify the type of rock according the three parameters, lithology, depth and alteration, as described above, and for the same classes of rock conditions one can correlate the number of probe holes to be grouted to meet any specified rate of inflow of ground water by using the charts shown in Figure 25.

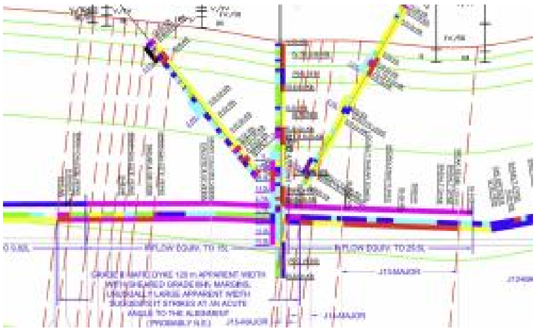

Figure 30 Part section of geological model close to location of shaft

By using site characterization and simple correlation as Peter Lumb taught one can compute estimates of the number of probe holes to be grouted and other items of tunnel performance and the result is an auditable estimate. With sufficient data and skill the results should be more useful than a few dozen Lugeon Values from packer tests.

GEO now requires all tunnel contracts to record performance data on a TDMS. Not only will the performance be available for inspection in real time but the valuable data will be available for purposes of such correlation.

5. LOTS OF MONITORING

6.1 Monitoring basement construction

Monitoring of construction works not only provides data, as recorded by the TDMS, but also plays an important role in control of the safety of the works. Large construction projects in densely developed urban areas have extensive monitoring. Such data is now reported electronically and can be readily studied and correlated. Not so long ago data was reported as hard copy. For example an 80 storey building was monitored during construction with many types of instrumentation including some 543 surface settlement markers and electro-levels inside running MTR tunnels. During 2 years there were 396,390 daily readings. From the settlement markers. Each month a dozen lever arch files of monitoring data were copied to all concerned.

The project was innovative. There were four “Mega-columns” each was founded on dense saprolite with a large shaft, see Figure 31. However sorting through all that data manually is a formidable task. Fortunately this project was one of the last that was reported by hard copy.

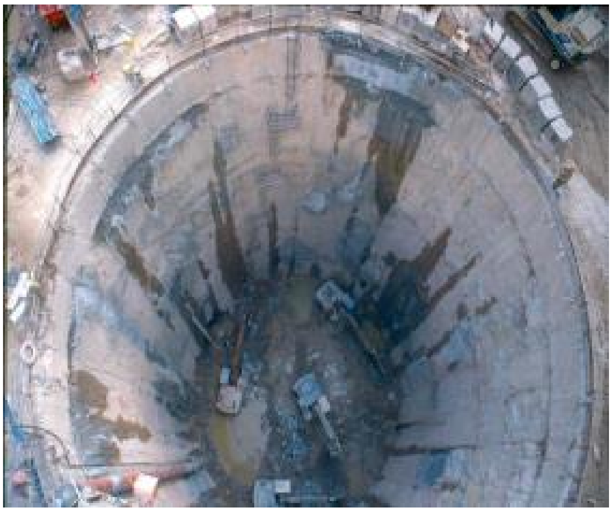

Figure 31 Foundation for one column of an 80 storey building

6.2 Monitoring driven piles

Steel H-piles have been driven for dozens of years. A lot of experience of driving piles has been accumulated. A long time ago the Hiley Formula for piles was developed. It considers the energy from the hammer as the weight times the drop. Impact on the cap of the pile loses a lot of energy. The formula is based on a simple model whereby the energy that is transmitted to the pile is equal to the force times the displacement. Displacement includes compression of the packing and the set of the pile.

On this basis, it is commonly held that the driving force, therefore the capacity, is inversely related to the set. The smaller the set, the harder is the driving. In order to develop a high capacity the pile is driven to a small set, and a large set is unacceptable.

These simplified assumptions worked well for many years and capacities were verified by static load tests.

Instrumentation has become a lot less expensive, piles can be monitored during driving with strain gauges that measure the driving force versus time at the top of the pile as well as the acceleration from which the displacement versus time can be computed during driving. This data can be analysed by a Pile Driving Analyser (PDA). A computer program CAPWAP can deduce the bearing capacity of the piles. The method has been extensively validated in Hong Kong, (Fung et al.2004).

PDA data for a project are shown in Figure 32. This data shows many piles with a driving force of 6000kN or more regardless of the set which ranges from 1 mm to 100mm per 10 blows of the hammer. This data is contrary to some of the prejudices, where by one might have expected the capacity of a pile driven to 25 mm per 10 blows to be substantially greater than a pile driven to say, 60mm, 80mm, or even 100mm per 10 blows. Monitoring can provide insight which sheds new light on old prejudices. In particular a simple model which fitted the performance for short piles fits the performance less well as the piles get longer. PDA is now regularly adopted as the criterion of acceptance of driven piles in Hong Kong.

Peter Lumb studied many driven piles and was well aware that the Hiley Formula was not appropriate for long piles, yet the industry still requires objective sets to be calculated for driven piles by using a modification of the Hiley Formula.

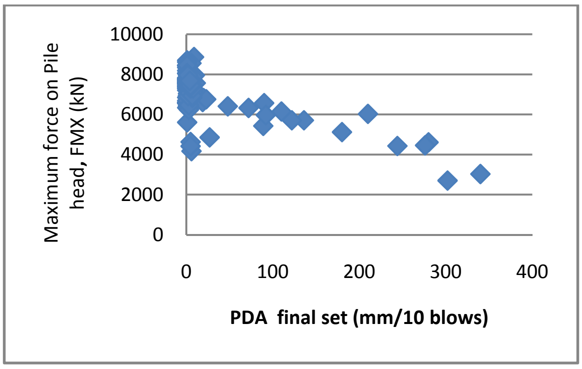

Figure 32 Data from PDA monitoring

6.3 Monitoring cut and cover tunnel construction

Although the cut and cover method has been used to construct tunnels for underground railways for many years, for example since 1976 in Hong Kong, there remain many uncertainties as to ground conditions and risks. When working some 15m or more below ground, safety is very important. Monitoring of deep excavations is primarily for safety and for control of the performance.

Therefore when a catastrophic collapse occurs the reasons should be understood, lessons should be learned and disseminated to prevent similar occurrences. Examples from the collapse of the Nicoll Highway in Singapore, (Magnus et al. 2005), are illustrated below. The site after the collapse is shown in Figure 33.

Figure 33 Collapse of cut and cover excavation alongside the Nicoll Highway, Singapore

The collapse occurred on 20th April 2004. In February 2004 there was a problem with the connection between the struts and the walers at a nearby section of the excavation. At 8am on 20th April 2004 the walers were distorted at the location where the collapse took place later that day.

The Site Engineer went into the site office and checked the real time monitoring of forces in the struts at that location. He was relieved that the forces in the struts were well below the "Alert" Limit. Secure in the knowledge that the struts were not over-stressed he went down the excavation to supervise the repair of the steel walers. Individual strut forces at the time are plotted in Figure 34 and show that the total measured force was 19094 kN whereas the designed total force was 28470 kN. A simple hand calculation shows that the total of the strut forces was barely equal to the active pressures on the walls and the inclinometers were showing that the two walls were moving towards each other. Any reduction in the capacity of the struts would have led to catastrophic collapse.

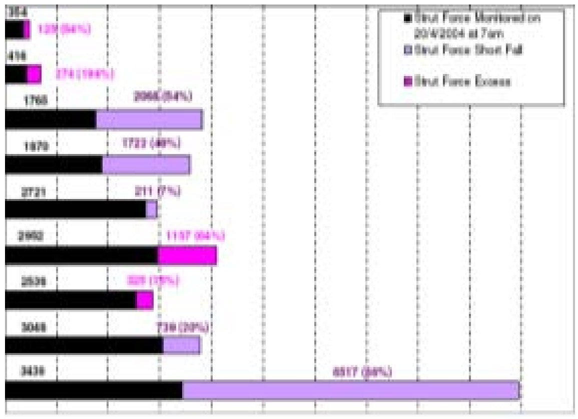

Figure 34 Forces in struts

Peter Lumb was an advocate of simple checks that can be done by hand.

The forces in the struts bear further examination. A basic assumption is that the strutting would behave elastically, i.e. the force in the strutting would be proportional to the compression of the strutting:-

$$\text{Compression of strut} = \frac{\text{Force x Length}}{\text{E x Area}}$$

$$\text{Force in strut} = \frac{\text{E x Area x Convergence}}{\text{Length}}$$

In Figure 35 the forces measured in a strut at the seventh level are plotted against time as a heavy line.

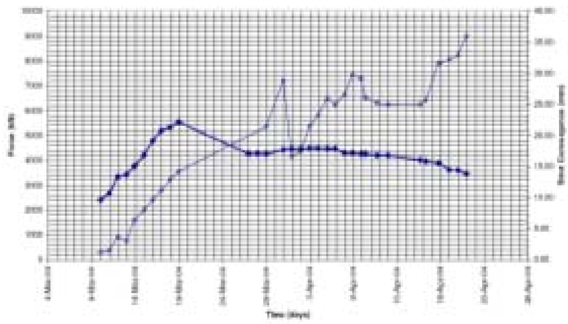

Figure 35 Plot of forces in strut and convergence between the walls versus time

On the figure the convergence between the inclinometers to either side of the location at the level of the strut is plotted as a fine line. The scale is chosen multiplied by ((E x Area)/Length) of the strut to compute expected forces in the strut. From 10th March 2004 until 19th March 2004 the two plots are parallel indicating that the strutting was performing elastically. After 19th March 2004 the plots are not parallel and the overall trend is that the walls converged by a lot more and instead of the force in the strut increasing proportionally it decreased. One can deduce that after late March 2004 the strutting was not behaving elastically as was expected.

Again this check is based on a simple calculation that can be done by hand as Peter advocated.

6.4 Back analyses were not carried out properly

On 23rd February 2004, almost two months prior to the failure data from inclinometers showed a maximum deflection of 152.9mm which was greater than the "Action" Limit and was considerably more than the Designer's estimate of 105.3mm. A back analysis was carried out by adjusting some of the parameters until the computed maximum deflection was 172mm. Profiles are shown in Figure 36.

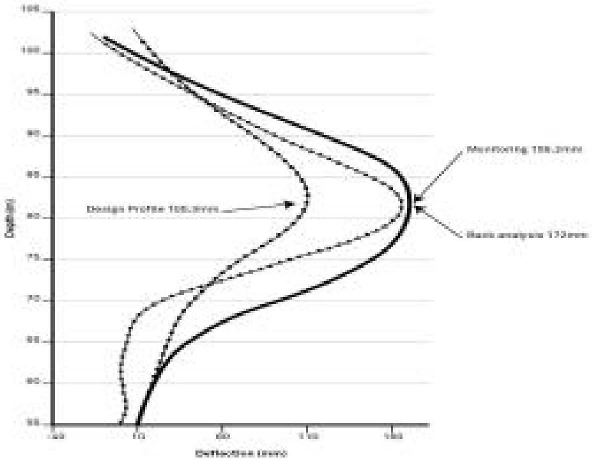

Figure 36 Profiles of deflection of the diaphragm wall

It is evident that whereas the maximum deflection computed by this back analysis was close to the maximum that was monitored, the deflections were not similar at other depths. Moreover the curvature computed by back analysis was far less severe than the monitoring results. The curvature reflects the bending moment in the wall and therefore the bending moment deduced by the back analysis was very much less than that would have been deduced from the curvature reported by monitoring the inclinometers.

Moreover, the back analysis did not match the inclinometer readings on the other side of the excavation excepting only the zero movement at the top of the wall and at the very bottom of the inclinometer, see right hand side of Figure 37. Some days later the displacement on the left hand side had well exceeded the back analysis as shown in the figure.

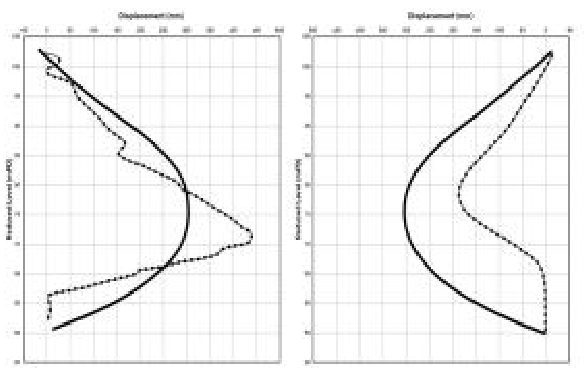

Figure 37 Comparison of back analysis with both sides of the excavation

6.5 Things still go wrong

Immediately after the collapse, one of the requirements of the authorities in Singapore was that designs based on analysis using PLAXIS should include two types of settings, Method A and Method B, and that the designer should justify which one would be adopted for the design. When assisting in checking, I asked a designer which method did he propose, he replied:-

"You chose."

Shortly afterwards the requirement was revised. Irrespective of the basis of the design assumptions, the designer should present analyses based on both undrained and fully drained behavior. A designer presented his analyses. The undrained case estimated the smaller capacity for the walls, the drained case estimated the greater capacity.

The tendered design adopted an intermediate capacity based on a coupled consolidation analysis. A dispute arose for which international experts were engaged. They carried out their own analyses and agreed that complete drainage would likely happen in this case. The reason why the designer's coupled consolidation analysis was almost fully drained but was so different from his drained analysis due to different boundary conditions in the two analyses. Had it been realized before construction that the coupled consolidation analysis estimated virtually complete drainage at each stage of excavation the dispute might not have arisen.

The initiation of the collapse of the Nicoll Highway was found to be an inadequate detail of the connection between struts and walers. Before the collapse it was realised that the connection was inadequate. It was changed and a piece of channel section was welded to the walers, see Figure 38.

Figure 38 Photo of the changed strut to waler connection

Four years later, a similar detail was adopted for an excavation also about 30m deep with ten levels of steel struts. The detail is shown in Figure 39. Instead of a channel section an angle section is adopted.

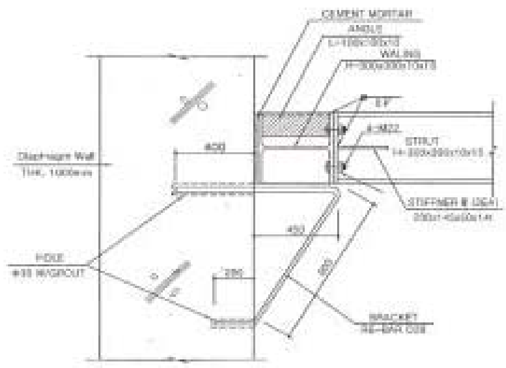

Figure 39 Detail of a strut to waler connection

Unfortunately the earth retaining structure where the detail was adopted collapsed although it is not known whether this detail contributed to the collapse or not.

6. LATERAL THINKING

Later in his career Peter Lumb became interested not just in applications of probability theory in soil mechanics but in pure statistics. He thought laterally. Geotechnical engineers of today apply their minds to other issues too as illustrated below.

7.1 Advanced modeling of slopes

A few years ago some slope movements were monitored and the mechanism of movement was discussed in a paper, (Yip 2001). Monitoring data from inclinometers was presented, see Figure 40.

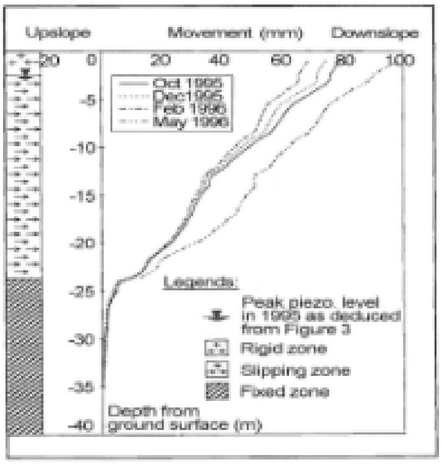

Figure 40 Inclinometer monitoring in a slope (After Yip 2001)

The author discussed various mechanisms of deformation which focused on solid body sliding on a basal plane. However the paper did not make an important observation that in the dry season from October 1995 to February 1996 the slope moved uphill. By May 1996, after the onset of the wet season, the slope had moved downhill again.

As more data became available from other sites this phenomenon, of seasonal movement uphill in the dry season and downhill in the wet season, was observed in some other slopes, (Cheuk et al. 2009). Data from another slope is shown in Figure 41.

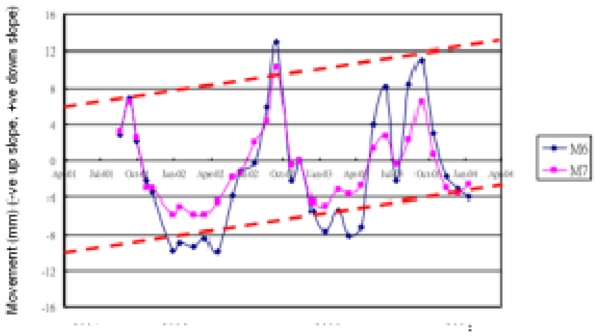

Figure 41 Seasonal movements of a slope

This data led to a line of research. A centrifuged model slope was subjected to wetting and drying, sees Figure 42, and slope movements uphill and downhill were observed, see Figure 43.

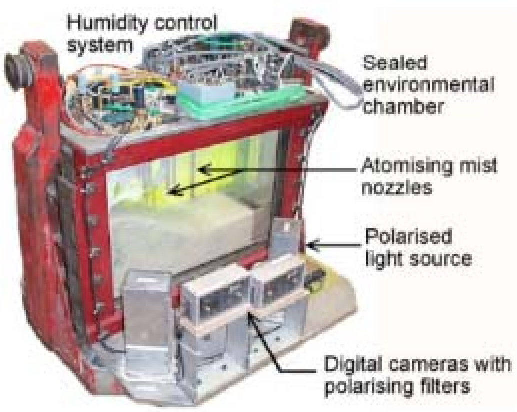

Figure 42 Cetrifuged model test on a slope subjected to wetting and drying (After Take and Bolton 2004)

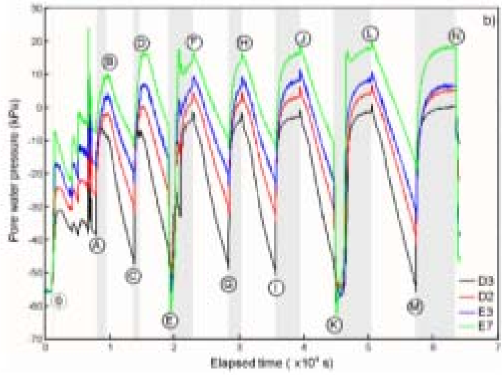

Figure 43 Movements uphill and downhill observed in a model slope subjected to wetting and drying (After Take and Bolton 2004)

Assuming that wetting and drying changes the isotropic pressure in the soil and that the deformation, as determined from the model, is mostly shearing up and down the slope, conventional numerical models, such as elasticity, could not replicate shear strain resulting from change of volumetric pressure. In order to numerically model the observed behaviour it was necessary to create an unusual relationship of transverse anisotropy as set out in the stiffness matrix as follows:-

$$\begin{bmatrix}\delta\varepsilon_{xx}\\ \delta\varepsilon_{yy}\\ \delta\varepsilon_{zz}\\ \delta\gamma_{yz}\\ \delta\gamma_{zx}\\ \delta\gamma_{xy}\end{bmatrix}=\frac{1}{E^{*}}\begin{bmatrix}1/\alpha^{2} & -\upsilon^{*}/\alpha^{2} & -\upsilon^{*}/\alpha & 0 & 0 & 0\\ -\upsilon^{*}/\alpha^{2} & 1/\alpha^{2} & -\upsilon^{*}/\alpha & 0 & 0 & 0\\ -\upsilon^{*}/\alpha & -\upsilon^{*}/\alpha & 1 & 0 & 0 & 0\\ 0 & 0 & 0 & 2(1+\upsilon^{*})/\alpha & 0 & 0\\ 0 & 0 & 0 & 0 & 2(1+\upsilon^{*})/\alpha & 0\\ 0 & 0 & 0 & 0 & 0 & 2(1+\upsilon^{*})/\alpha^{2}\end{bmatrix}\cdot\begin{bmatrix}\delta\sigma'_{xx}\\ \delta\sigma'_{yy}\\ \delta\sigma'_{zz}\\ \delta\tau_{yz}\\ \delta\tau_{zx}\\ \delta\tau_{xy}\end{bmatrix}$$

By adopting these new parameters computations can be carried out which reproduce reversible shear deformation due to wetting and drying as shown in Figure 44. The field observations show significant reversible deformation and some irreversible deformation. The numerical model would have to be refined to include plastic deformation to fully replicate the field data.

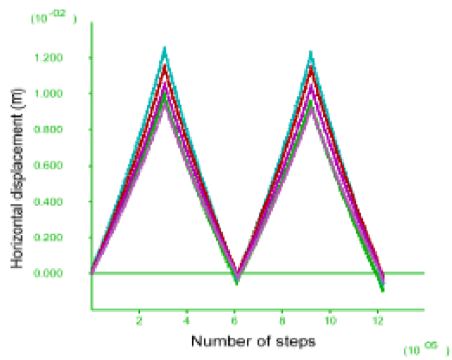

Figure 44 Numerical modeling of cyclic movement of a slope (After Cheuk et al. 2009)

7.2 Capturing topography in 3-D and visualisation

New technology includes taking pictures from a low flying aircraft and developing digital images of the topography. An oblique image obtained by using Light Detection and Ranging Technique (LiDAR) is shown in Figure 45. LiDAR is a new terrestrial laser scanning technique for quick, low-cost, and vast area topographic surveying applications. It provides high accuracy topographic data for engineering modelling and design works. Figure 46 shows a 3-D digital ground model generated from LiDAR data for modelling natural hillside failure and design of mitigation works see Figure 47.

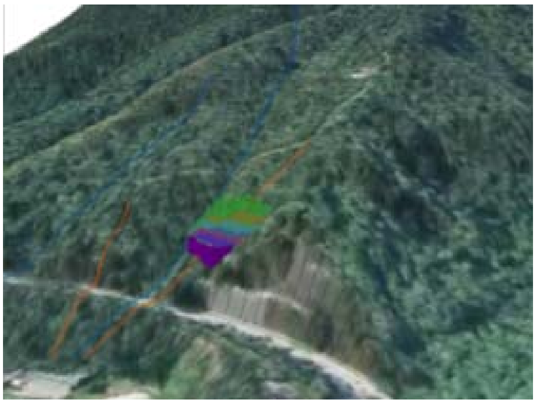

Figure 45 Oblique digital image

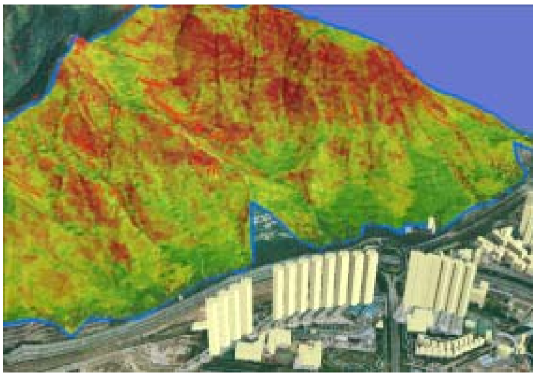

Figure 46 3-D Digital Ground Model Generated from LiDAR Data

The data can be used, for example, for 3-D dynamic mobility modelling to assess the run-out of potential debris flow and to evaluate the positioning of the proposed natural terrain hazard mitigation works, see Figure 47.

Figure 47 Numerical modeling of debris run-out

7.3 Conventional 3-D modeling

Numerical modelling of geological profiles has been widely used and such surfaces can be very complex. However standard techniques of contouring rarely plot outside the data base. For example the highest point in the surface is often a pinnacle centered on the highest level in the data base and the lowest point is at the bottom of a deep pit, see Figure 48, whereas the reality might include ridges joining inferred pinnacles and troughs connecting inferred pits.

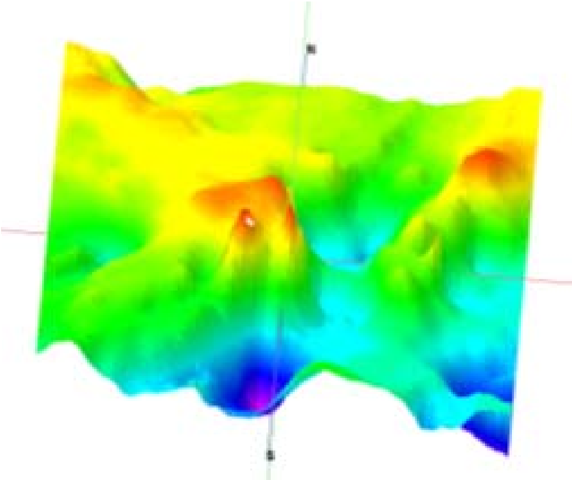

Figure 48 Computer generated profile of rock head showing pinnacles and pits

These simple contouring programs provide no interpretation of common features such as ridges and gullies and other lineaments. However more complicated software is available.

Large computing capacity permits very large models to be developed, and for associated designs such as alignment of tunnels, see Figure 49.

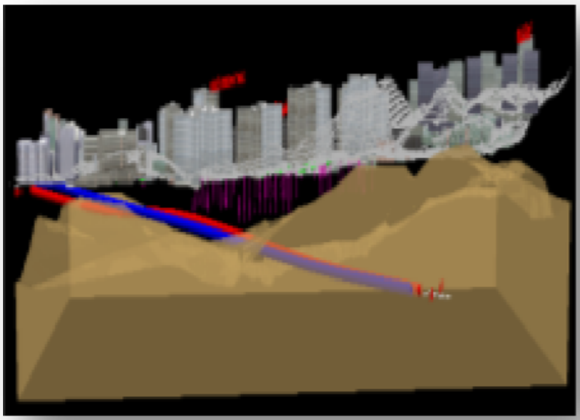

Figure 49 Tunnel Alignment and Bedrock

7.4 Geographic Information System

Geographic Information System (GIS) was developed for handling large quantities of spatial data such as geological data. GIS is now widely used by geologists and geotechnical engineers. For example data about slopes and catchments are handled by this means, see Figure 50.

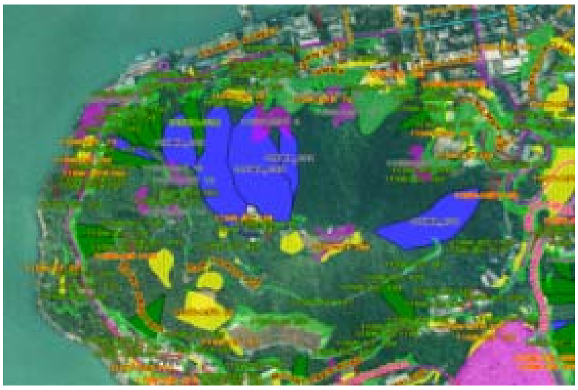

Figure 50 GIS data on catchments

GIS data can also be used for geotechnical design. For example predictions of the effects of blasting in a tunnel, see Figure 51.

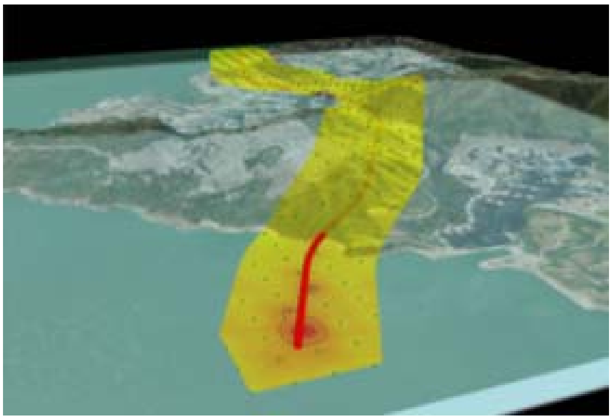

Figure 51 GIS used for basting impact zone

GIS has far more potential than geotechnical applications. It includes Building Information Management and other examples include simulation of traffic noise studies for environmental engineers, see Figure 52. Applications of GIS show lateral thinking that is comparable with Peter Lumb's interest in pure statistics.

Figure 52 GIS for simulation of traffic noise

7. CONCLUDING REMARKS

Professor Peter Lumb was a pioneer in the early days of geotechnical engineering in Hong Kong. This paper reviews changes in geotechnical practice, in and around Hong Kong, since his retirement and shows remarkable developments and some folly. What would he think of his legacy? Would he be disillusioned by folly or would he have taken satisfaction to see that, in many instances, his legacy lives on.

There are a number of valid successors following in Peter's footsteps. This paper has drawn extensively upon the work of many good geotechnical engineers and is dedicated as a tribute to all of the geotechnical engineers, engineering geologists, geologists and other people who have made the name of Hong Kong synonymous with ground engineering. There are too many to single out individually.

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