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Case Histories of Failure of Deep Excavation. Examination of Where Things Went Wrong: Nicoll Highway Collapse, Singapore

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CASE HISTORIES OF FAILURE OF DEEP EXCAVATION EXAMINATION OF WHERE THINGS WENT WRONG: NICOLL HIGHWAY COLLAPSE, SINGAPORE

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ABSTRACT

During the construction of Contract C824 of the Circle Line in Singapore, on 20th April 2004 an 80 m long section of excavation, 30 m deep, totally collapsed. The resulting crater was as deep as 15 metres and was more than 100 m in diameter. Six lanes of the adjacent Nicoll Highway subsided by as much as 13 m. Four construction workers were killed. Fortunately no vehicle was involved.

A Committee of Inquiry was established and held hearings from August 2004 until March 2005. The findings of the Committee were published in May 2005. These identified the causes of the failure and made recommendations concerning safe practices for deep excavations in the future.

The paper includes an account of the events leading up to the failure, the identification of the causative factors, and the reasons for the total collapse.

There were many factors which caused the initial failure and the subsequent overall collapse. Although the trigger for the failure was found to be inadequate detailing of the connections between the steel struts and the steel waling beams, many contributory factors led to the whole structural system being unable to cope with the failure and the systematic failures in the management system. In addition to prosecution, the Authorities in Singapore took cognizance of the lessons learnt and took immediate follow up actions. These included immediate checking of the design of all similar deep earth retaining structures. Interim Guidelines were introduced which have since been followed up with revised standards such as independent checking of temporary works design, independent contractors for instrumentation and monitoring, and upgrading the factors of safety for deep temporary excavations to be the same as those for permanent works.

The paper concludes with observations of what has happened in the subsequent seven years. Whereas a number of controls on procedure have been tightened, similar mistakes in detailing and lack of comprehension of the computer analyses have been observed and failure of a similarly deep strutted excavation occurred only three years later, but not in Singapore.

INTRODUCTION

Engineering has developed through innovation. Much of the work of Ralph Peck and Clyde Baker has been innovative, and successful. On the other hand much has been learned from disasters, such as the infamous wrought iron Tay Bridge in Scotland and the suspension bridge over the Tacoma Narrows in U.S.A. The massive collapse of the Nicoll Highway in Singapore was a milestone in the engineering of deep Earth Lateral Support (ELS) structures. Singapore was rightly proud of its extensive underground railway system and, prior to 2004, it’s record of safety with very few fatalities. In Singapore diaphragm walls with temporary steel strutting have been in use for metro rail stations since 1978. Methods of design have been tried and tested over the years. This paper addresses what went wrong to result in such a massive failure and what needed to be done to ensure that such failures do not happen again. After the collapse certain measures were implemented. That was about 8 year ago. The measures have been taken up by the industry but have they been effective. The paper concludes with observations of what has happened in the subsequent seven years. Whereas a number of controls on procedure have been tightened, similar mistakes in detailing and lack of comprehension of the computer analyses have been observed and failure of a similarly deep strutted excavation occurred only three years later, but not in Singapore.

THE PROJECT

The site was formed by reclamations about 30 years ago when dredged sand, about 3 m to 5 m thick was placed on a shallow sea bed over about 30 m depth of very soft to soft marine clay. Consolidation under the fill is not fully complete and residual pore water pressures of up to 3 m have been measured. Beneath the marine deposit there is a few metres of alluvial soils followed by weathered" Old Alluvium" which is a weakly bonded sandstone that is weathered at the top to a dense sandy soil.

Prior to the collapse, a cut and cover section of tunnels was under construction for the underground railway system of the Mass Rapid Transit System (MRT) on behalf of the Land Transportation Authority (LTA). The site was located within an open recreational area alongside the Nicoll Highway which is a dual three lane urban road, see Plate 1.

Plate 1. Photograph of the Collapse

The excavation work was in progress. Concrete diaphragm walls had been constructed about 20 m apart and, as excavation advanced, steel struts were installed to support the lateral earth pressures. At this location nine levels of struts had been installed and the excavation had reached a depth of about 30 m and struts at the tenth level were about to be installed. This form of construction has been used extensively in Singapore for construction of the MRT more than 25 years. Many contractors are experienced at diaphragm wall construction and re-usable steel strutting with bolted connections can be hired from local suppliers. MRT underground rail structures have been built successfully in reclaimed land since the early 1980's to depths of about 18 m to 20 m. However excavation as deep as 30 m in soft clay had not been carried out before.

Prior to bulk excavation, jet grout piling (JGP) had been installed in two horizontal layers to act as buried strutting. These were located excavation stages 9 and 10 and below the final excavation level respectively. Jet grouting is commonly used generally and has been used as buried strutting on previous projects in Singapore. After installing the 9th level of struts the excavation had been delayed by hacking out the upper layer of jet grout and by cutting an "eye" in the wall of the adjacent shaft in readiness to hand over the site, within four weeks' time, to the track laying contractor.

THE COLLAPSE

The collapse was dramatic. A length of 80 m of the excavation totally collapsed. The two diaphragm walls converged with destruction or gross displacement of the nine levels of steel strutting. The ground outside the excavation subsided forming a crater with a diameter of about 100m and a maximum depth of about 13 m. See Plate 1. The collapse included a section of the six-lanes of the Nicoll Highway Fortunately no vehicle was involved but four site workmen lost their lives.

The commercial loss was substantial because as a consequence of the failure the tunnel and adjoining station were abandoned.

Early press reports included "gas explosion collapses tunnel". It was soon thought that the collapse of the excavation caused the rupture of the gas main under the adjacent road.

The first lesson learned is that Press reports can not always be relied upon.

Informed opinion in the technical press severally attributed to collapse to base failure in soft clay, improper use of a computer program, and inadequately designed walls respectively. Base failure as a cause was later withdrawn but improper use of a computer program and inadequately designed walls were found to be only part of the story. Informed opinion is necessarily based on the information that is available and premature opinions should only be given cautiously.

EVENTS LEADING TO THE COLLAPSE

In February 2004, two months before the collapse, at an adjacent section of the excavation there had been problems with the connection between the struts and the walers which led to retrofitting a modified stiffener to the waler. At about the same time the basis of the design of the temporary lateral earth support system was seriously questioned. An expert for the Land Transport Authority (LTA) said that the computer program had been used incorrectly and that as a result the deflections and bending moments in the walling were under-estimated. The Contractor disagreed.

Early on 20th April 2004 similar distress was observed at the walers at the 8th and 9th levels at the location of the collapse. A team of workers were mobilized to reinforce the walers by pouring concrete into the upper half of the "H" section. The Site Engineer checked the computer based monitoring and was reassured that the forces in the struts at this location were below the alarm levels. During the morning the situation became worse. The distortion of walers became worse. The workers abandoned the 9th level and set to work on the 8th level. By mid afternoon the order was given to evacuate the site. Unfortunately an overall collapse occurred and four workers were killed.

About 80 metres length of the diaphragm walls converged to meet each other. The steel struts were destroyed. Adjacent to the walls the subsidence was about 13 m and the visible settlement extended to a radius of about 50 m, see Plate 1.

PUBLIC INQUIRY

A Committee of Inquiry was established. Public hearings were held from May 2004 until December 2004 and the Final Report was published in 10th May 2005. The Report of the Inquiry, Ref 1, is about 1000 pages long. It identifies lots of things that went wrong. Other things could have also been wrong but there was inadequate information to be able to determine if they did go wrong or not. The collapsed site was not subsequently excavated so physical evidence was lacking and, prior to the failure, only selected sections had been monitored.

The Committee decided that there was a host of causative factors. However they ranked the factors as summarized below. It was recognized that civil engineering relies on a level of redundancy such that, if one element fails the forces involved are-distributed and total collapse requires more than one element to fail. The Committee decided that design errors resulted in the failure of the 9th level strutting and the ensuing total collapse resulted from inability of the retaining system as a whole to resist the redistributed loads arising from failure of the strutting at the 9th level.

<u>Major causes of the collapse included the following</u>:

- Errors in the design of the strut to waler connection,
- Two erroneous back analyses,
- Deficient monitoring at the site.
- Incorrect use of the computer program.

<u>Contributory causes were listed as follows</u>:

- Inconsistencies between design criteria and codes;
- Insufficient toe embedment for hydraulic cut off;
- Special geometry not taken into account;
- Cable crossing disrupted diaphragm walls & JGP slab;
- Inadequate appreciation of complex ground conditions;
- Inappropriate choice of permeability for OA;
- Delay due to cutting walls for tunnel;
- Large spans left un-strutted for a long time;
- Loss of preload in struts levels 8 and 9;
- No check of one strut failure in back-analysis;
- Work did not stop in face of warnings on site;
- Failure to implement risk assessment;
- No independent design review;
- Weakness in the management of construction changes;
- Instrumentation system not effectively used.

There were many issues. There were procedural faults, technical faults, urgency to complete two months work in one month in order to meet a hand-over date, and overall air of complacency.

INITIATION OF FAILURE

In February 2004, two months before the collapse, walers between struts and wall panels at a nearby location were found to have buckled. As a consequence the design of the connection between the struts and the walers was changed. Pieces of steel channel section were added to strengthen the connection, see Plate 2. Unfortunately the strengthening did not provide adequate capacity. Subsequent laboratory tests on perfectly constructed samples showed about half the capacity, see Figure 1. Moreover whereas steel normally strain hardens and has marginally increased capacity after initial yielding, the connection exhibited brittle failure with massive reduction in capacity after first yielding.

Plate 2. Photograph of Revised Stiffener

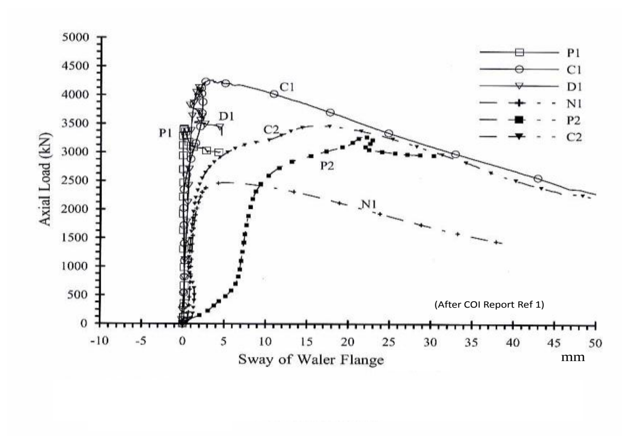

Fig.1. Test results for capacity of connection

At about 8 am on 20th April 2004, site staff noticed buckling of walers at the 9th level and set about trying to strengthen them. However during the day the conditions worsened. Walers at the 8th level buckled and at 3 pm total collapse occurred.

Neither the original design nor the revised design for the connection was properly carried out.

COMPUTER MODEL

The tender design for the diaphragm walls and the strutting was based on analyses carried out using the Katsetzu5 computer program. During construction the computer program PLAXIS was used initially to check the Katsetzu5 program but then replaced it. PLAXIS is now commonly used but had not been used for LTA's projects in Singapore at the time. The program can model the soil and associated structures and the excavation sequence.

The majority of the soil supported by the diaphragm wall was soft marine clay which would not drain significantly during the works. Therefore it was rightly assumed that the soft clay would be undrained. PLAXIS allows input of undrained shear strengths directly, Method B, or input of effective stress strength parameters c' and φ', whereby the shear strength is calculated using the Mohr Coulomb model with an undrained setting, Method A. The undrained setting, with no change of volume of the soil, results in no change in the isotropic stress, p', and an over-estimate in the strength of a normally consolidated clay. The manual recommended the use of Method A. The error is about 18% over-estimate of the strength of one-dimensionally consolidated clay Figure 2. The effect is an underestimate by only about 4% underestimate of the force applied to the wall for any one stage but for multi-stages the cumulative effect is greater and, for example, using Method A for this project the computed displacement of the wall is about half of the value obtained by using Method B. The bending moment in the wall is similarly under-estimated.

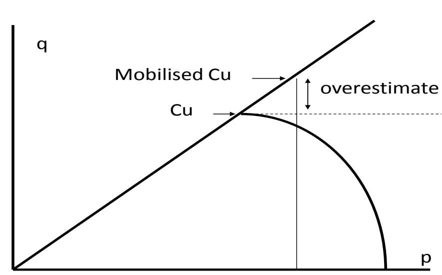

Fig. 2. Overestimate of strength when using Method A

As a consequence of using Method A, the diaphragm walls were weaker than they should have been. However, during the day of the collapse, inclinometers showed that the walls only developed one plastic hinge and not two hinges that would be required for failure of the wall.

The lesson learned is to understand what the program does and which method to use. One should not follow the manual without making sure that the recommendation applies to the particular application.

Checks on the design and monitoring during construction should have picked up this error. The error was picked up. Some three months before the failure, at two other locations of this project, a university Professor pointed out that Method B should have been used. Based incorrect interpretation of monitored displacements during early stages of excavation of wall panels nearby, the designers argued that Method A could be used. The Professor and LTA's senior technical staff pointed out the severity of the problem but senior managers took no action.

Lessons learned are that monitoring should be interpreted properly and that senior management should heed advice from senior technical staff and from expert advisors.

MONITORING

An essential part of site safety is monitoring the performance of the works. An instrumentation station was located in the middle of the sections of the excavation that collapsed. The instrumentation included two inclinometers that were used to monitor the lateral displacements of opposing wall panels and included strain gauges on all levels of struts to measure the forces in the struts at the same location.

Inclinometers located inside walls or in soil adjacent to wall panels are used to monitor displacements of the walls. From the inclinometer profiles one can determine the radius of curvature at any given time from which it is possible to determine the bending moment in the wall panel. Inclinometer results showed that a plastic hinge developed in the wall panel at the location of the failure some three weeks months before the collapse, see Figure 3. The Alarm Level for the displacement was exceeded and it was relaxed twice, without recognizing the distress of the wall panel at the time. On each occasion a back analysis was carried out which matched the maximum displacement but did not match the deflected radius of curvature. In the back analyses, the soil strengths were reduced to develop more pressure on the walls and induce more displacement. This also resulted in increasing the calculated forces in the struts which did not represent the observed reduced forces in the struts. A correct back analysis would have identified that there was a problem with the struts.

Alert, Action and Alarm levels (AAA) of maximum movement are relevant to limiting ground movement and protecting the surrounding ground and adjacent property. Such limits do not relate to the performance of the wall in bending for which a limit on the radius of curvature would be appropriate.

Lesson learned is to understand the monitoring and to set appropriate the AAA limits for the capacity of the structural members as well as for protection of property.

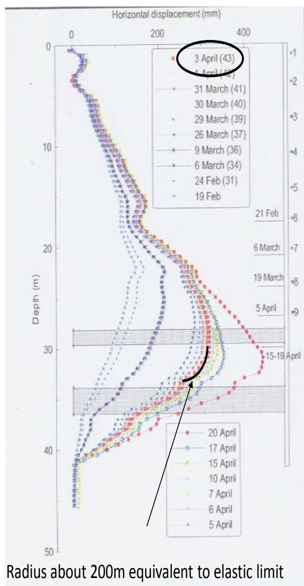

Fig.3. Inclinometer readings

COMPARISON BETWEEN CONVERGENCE OF WALLS AND FORCES IN STRUTS.

More insight can be obtained from the inclinometer readings. The two inclinometers were located in wall panels on opposite side of the excavation and the sum of displacements for the two inclinometers is the convergence between the two wall panels. Under normal operations strutting should work within the elastic range and therefore the convergence between opposite wall panels and the forces on the intervening struts should be proportional to the stiffness of the strutting system. By comparing the two sets of data, for struts at level 7 as shown in Figure 4, although there is scatter in the convergence data, there is an overall trend of increasing convergence whereas the force in the strut does not change appreciably during late March 2004 and reduces during the three weeks in April 2004. Although the data is not precise, the trend is evident that the forces in the struts were not increasing as they should have done. The assumption in the back analyses that the strutting was behaving elastically was wrong and had the measured forces in struts been used as input data to the back analyses a proper back analysis of the wall could have been achieved.

Lesson learned is to conduct back analyses which properly match all of the relevant monitoring results and not just one result.

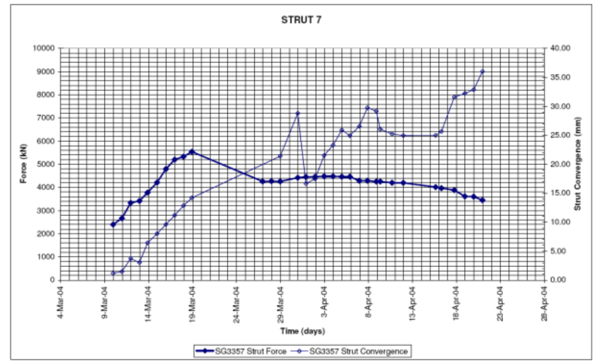

Fig.4. Force v Convergence for Strut at Level 7

LOSS OF PRE-LOAD

The design required the struts to be pre-loaded to 75% of the design maximum force and locked off. Forces in struts were measured and reported automatically every hour. The record of forces in the strut at level 9 is shown in Figure 5. Within one hour after lock-off the load had dropped to only 20% of the required pre-load. A similar result was recorded for struts at level 7 some two months before the collapse and again at level 8. One month before the collapse.

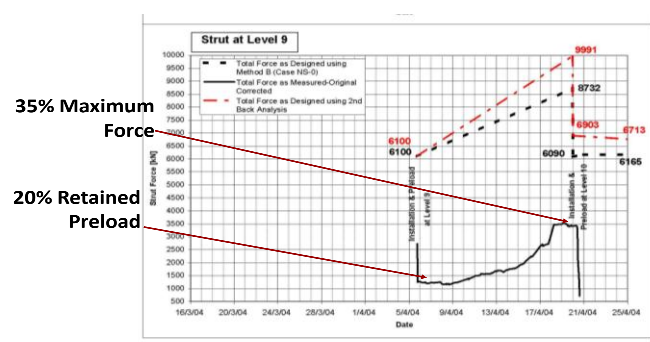

Fig.5. Force v Time for Strut at Level 9

The AAA limits were based on not over loading the struts. Because the AAA limits were set with a maximum force commencing at 50% of the design load no warning was issued. Just before the collapse the readings had not exceeded 35% of the design value. Moreover, it was not observed at the time that the force in the strut was only 35% of what it should have been and therefore the strut was not providing the support to the wall that was intended. At this stage the total forces provided by all 9 levels of struts on the wall was 67% of the design value which means that the wall was barely supported.

The lesson learned was to not only consider the maximum force in the strutting in order to prevent overload of the strutting but also to make sure that the forces supporting the wall were sufficient to provide the required factor of safety.

Importantly, the monitoring data shows inadequate performance of the strutting but the monitoring data was not properly studied and the inadequate performance of the strutting system was not noticed.

OTHER ISSUES

There were several other technical issues that came to light. For example the data from strain gauges were not properly corrected when strain gauges malfunctioned. Some questions could not be answered because of lack of evidence. For example did the JGP work, were the depths of embedment of the wall panels to either side of the instrumented panels adequate?

Some effects were not studied. At the location of the failure the depth to firm ground and the toe levels of the walls varied, the wall panels varied in thickness, and at one end they were restrained by the shaft for the bored tunnels. High voltage electricity cables crossed the site, and beneath the cables the diaphragm wall panels were omitted and the soil was supported by lagging, see layout in Figure 6. The analysis assumed plane strain uniform symmetrical cross section.

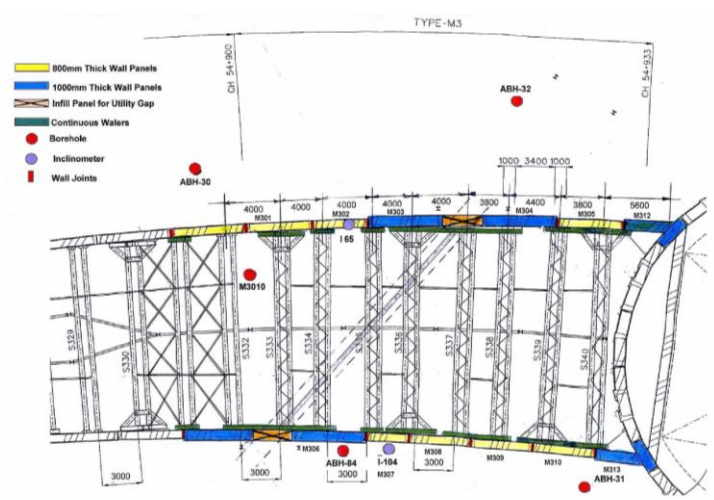

Fig.6. Layout of Area

ACTIONS TAKEN

Immediately after the collapse, LTA appointed independent checkers of the design for all then current temporary works, LTA commissioned re-examination of all temporary works under way, and LTA issued revised design principles for temporary retaining walls. Temporary works were to be designed to same standards as permanent works. Building and Construction Authority (BCA) required immediate check on all uses of PLAXIS.

In May 2005, BCA promulgated guidance notes to concerned parties, Ref.2 which set out new standards for site investigation, design and construction of deep earth retaining structures.. Major changes are advising that temporary works should be designed to the same standards as permanent works, checked independently. The areas covered include adequacy of site investigation, codes, standards, extent, groundwater conditions and existing building conditions. Issues of design include guidance on factors of safety, soil parameters, water pressures, robustness, numerical modeling, sensitivity analysis and jet grouted piles. Issues during construction include multi-tier level monitoring, design review, independent checks, site inspection an approval and instrumentation and monitoring

LTA has adopted similar procedures. These are more specifically geared towards underground railway construction and include, in addition, details of global stability checks, full consideration of failure of one level of struts, the use of software and submission of both drained and undrained analyses, proper back analysis, as well as maximum allowable movements and assessment of buildings and utilities. Monitoring is to be carried out independently of the Contractor. The registered Professional Engineers for design and for supervision shall be independent of the Contractor. Deep temporary earth retaining structures shall be designed to the same factors of safety ad permanent works. AAA limits shall be adopted for stages of excavation and not applied globally.

LTA appointed consultants to perform independent reviews and checks for the completion of the Circle Line and procures instrumentation directly and not via the main contractor.

The office of the Building Control Unit in LTA has been closed and now BCA administers Building Control for LTA construction projects.

HAVE THE LESSONS BEEN LEARNED?

Fundamentally the same mistakes are being made, I quote two examples.

The initial failure of the Nicoll Highway was initiated by in adequate design for the connection between the struts and the waling. An appropriate detail, hat has been used for many years is to provide stiffener plates welded between the flanges and to the webs for the walers and for the ends of the struts. Such plates provide a large bearing area for the connection and restrain the waler from shear buckling that occurred in this case.

In September 2007, only three years after the collapse of the Nicoll Highway, there was a collapse of a deep excavation in another city. The excavation had several similarities. It was 30 m deep in recent alluvial deposits with diaphragm walls, in this case terminating on rock just above and below the final excavation level. Ten levels of steel strutting, with king posts, with splays and steel walers were adopted in a similar arrangement. The connection between the struts and the walers were provided, with stiffeners made from steel angle section, see Figure 7. Channel sections were not used but the detail resulted in the same fundamental lack of shear stiffness. Conclusions about the cause of failure are incomplete because of lack of evidence.

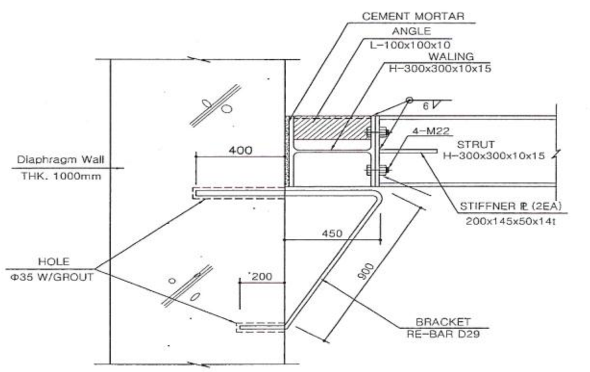

Fig.7. Connection Strut to Waler

A number of courses have been provided in Singapore on the use of PLAXIS and PLAXIS is now used by many designers. Even more sophisticated applications are in use such as coupled consolidation analysis which models transient seepage of ground water and consolidation of the soils. The designer for a deep excavation performed a coupled consolidation analysis which he intended to rely upon for his design. In accordance with local requirements the designer also performed both undrained and drained analyses. The results from the coupled flow analysis were found to be between these two limiting cases.

The difference between the coupled consolidation analysis and the drained analysis became an issue of dispute. Experts were engaged from overseas. Days before the hearing the experts agreed that, due to high permeability of the ground, the coupled consolidation analysis output showed almost complete consolidation and ground water pressure was very close to the steady state seepage case. The difference between the coupled consolidation case and the fully drained case was due to different boundary conditions adopted in the analyses. This had not been recognised by the designer from the time of the design until the experts pointed it out several years later. This is only one example of the use of a powerful computer program for which the numerical computations had not been fully understood.

OBSERVATIONS

The process for deep excavations involves design, design checks, monitoring of the works, setting and observing limits, back analysis of performance and verification of design assumptions. Usually several parties are involved. Proverbially the industry applies “belt and braces”. Failures should not happen. It is often observed that, in the case of failures of structures, the main elements are usually sound, it is the connections that fail. For deep excavations, even if a failure of a component occurs, the system should be robust enough to withstand the failure of a single component.

As a general observation, it is my opinion that downfalls arise from inadequate skills or experience being deployed. Attention to detail is important to prevent failure of individual components especially connections between otherwise adequate members. Lack of appreciation of fundamental concepts, such as not considering whether steel strutting performs elastically the force in a strut is proportional to the convergence between the ends of the struts, is also commonplace. Plenty of engineers understand how sophisticated computer programs work but their skills are not always deployed and I come across many cases where the user of a program is less skilled and does not understand the results.

In this case, the Authorities have taken action in Singapore to strengthen the system but, some lessons are not learned and some mistakes are repeated.

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