Lessons Learned from Failures of Some Housing Projects

S. S. Gue, S. S. Liew & Y. C. Tan
Gue & Partners Sdn Bhd, Malaysia

1 INTRODUCTION

Sometimes, failure can occur due to wrong decision without adequate and thorough thought. One good example is the Y2K problem, in which a mistake made few decades ago using 2 digits year instead of 4 to reduce the extra memory required for the extra 2 digits. This mistake has cost hundred billions of dollars to remedy the serious shortcoming and the figure is still counting. The same sometimes happens in civil engineering.

A number of failures of housing projects in Malaysia have been highlighted by media. Plate 1 shows one of the many news exposures. These failures have resulted financial loss and delay to all parties, particularly the innocent house buyers. If the failure occurs after occupancy, it would be a hazard to public safety and could also be a professional liability. Such news also has some negative impacts to the image of engineers in the public perception. This paper intends to share with engineers through these lessons as a good guidance to practice professionally and to upgrade the quality of professional service.

All these failures can be avoided if proper care has been given during planning, subsurface investigation, analyses, design and construction. Specialist input where necessary should be acquired to assist in a project.

The need of specialist input is very often omitted due to over confidence on ones’ previous experience on different ground conditions. Therefore, many projects suffered due to lack of appreciation on the ground condition and the inappropriateness of the geotechnical design by inexperienced professionals. Usually, when things go wrong, the costs of remedial works are hundred folds of the consultant fee for a specialist inputs.

This paper uses three case histories to illustrate the causes of fairly common occurrence in the building industry and hope that these lessons could be learned by all relevant parties (i.e. developers, consultants and contractors) involved in housing development to prevent recurrence.

Plate 1   Structural Distortion
2 THREE CASE HISTORIES OF FAILURES

2.1 Case One: Excessive Foundation Settlement & Short Piles

This case involved some 40 units of single-storey terrace houses and 70 units of double-storey terrace houses, in which platform settlement and structural cracking were observed. Plates 2 & 3 show the voids underneath the ground beam and the cracks at the ground beam respectively after fill settlement.

The affected buildings were on filled ground of varying depths up to 20m overlying the undulating meta-sedimentary formation. The platform was mostly filled using the cut materials with significant content of boulders, which subsequently posed problems to pile installation. The existence of boulders in the fill has been confirmed during the additional subsurface investigation, in which a trial pit done indicated that a boulder of about 1m in size was discovered. The subsurface investigation also revealed that the platform was not properly compacted to the normal standard of engineered fill.

The weathering profile is fairly consistent to the topographical profile of the original ground as shown in the borehole logging. The scattering SPT-N profile of the boreholes in the fill material with some very low N values at various localized depths indicated poor compaction of the fill.

There were two piling contractors, namely Contractor A and Contractor B, carrying out the pile installation at this site. The recorded pile penetration by the two contractors was very much different. Figures 1 & 2 show the pile penetration length record by the two piling contractors. The recorded pile penetration by Contractor A appears to be very uniform regardless of the variation of piling platform and the original topography. However, the piling penetration records by Contractor B shows a fairly consistent pattern with the weathering profile of the ground. The high strain pile tests carried out during the investigation on the piles installed by Contractor A have confirmed short piling.

In order to investigate the inconsistency of the pile penetration and to verify the short piling interpreted from the dynamic pile testing, some piles were selected for extraction. The pile extraction has confirmed short piling. Plate 4 shows the pile extraction. There was no sign of welding at the tip of the extracted piles, which should have for longer pile penetration. Plate 5 shows the pile tip condition.
Excessive platform settlement occurred a few months after completion of the earth filling and causing serious structural cracks on the newly constructed ground beams, stumps, ground floor slabs and the structural frames. More serious distresses were observed at units on thicker fill. Crack gauges and settlement markers were installed during the investigation by the authors to monitor the changes of crack width and platform settlement with time respectively. The results of monitoring confirm the settlement and cracks were active. The observed crack pattern on the wall was mostly diagonal and initiated from the corners of the door and window openings that were the areas of stress concentration when distortion of the structures occurred. Cracks on the structural frames were mainly located at the beam-column connection and the mid span of the beams, which have maximum flexural stresses.

2.2 Case Two: Platform Settlement and Wall Collapse

This case history was featured with many geotechnical failures in the infrastructure works and the buildings. The project site was located at a previous rubber estate with undulating terrain of meta-sedimentary formation and a few numbers of natural water streams within the project site. As the development area was very large in lateral extent, the earthwork design concept was based on cut and fill balance and to minimize rock excavation as bedrock was commonly expected at shallow depth below the undulating original terrain and has a fairly consistent weathering profile. Therefore, the earthwork platform was finally completed with various fill thickness and earth cutting. The maximum fill thickness in this area was about 32m. Unfortunately the filling areas were not properly compacted as engineered fill. Many problems were subsequently surfaced up as a result of fill settlement, which was largely attributed to poor compaction and collapse compression after saturation with prolonged rainfall and rise of groundwater level.

There were about 200 units of double-storey terrace houses in one of the development phases suffering from significant fill platform settlement. These buildings were supported on 125mm×125mm timber piles. The non-suspended ground floor slab became suspended after the platform settled and caused the ground floor slab to sag. The partial friction between the four sides of the floor slab and the ground beams restrained the slab from settling. Figure 3 shows the schematic diagram of the sagging ground floor slab and Plate 6 shows the settling floor slab. Voids underneath the ground floor slab was suspected during final finishing of the floor tiles and was further confirmed after hacking the suspected slabs for inspection as shown in Plate 7. The maximum gap between the center of the sagging slab and the settling fill platform as measured at site was about 150mm. Therefore, the total settlement of the platform at the time of investigation was estimated to be about 225mm, assuming the buildings did not settle. The pattern of platform settlement coincided very well with the original topography, in which larger platform settlement occurs at the thicker fill areas.
Another concern of the associated platform settlement was the potential negative skin friction developed at the piled foundation, although at the time of investigation there were no serious distresses or settlement observed on the structural frame except some vertical tension cracks between the units on split levels platform. The review of the topography survey of the affected units and the piling records reveals that some of the installed foundation piles terminated within the fill and some penetrated few meters into the original ground. It appeared that the piles were installed to predetermined length. The telltale sign of the vertical tension cracks on the external walls of buildings on different platform levels had indicated potential future excessive differential settlement at areas where piles were installed within fill of varying thickness. For those piles penetrated into original ground, predominantly end bearing condition could be expected, in which negative skin friction due to platform settlement could overstress the foundation piles beyond the allowable design stress level of the piles but this has yet to reach structural failure at ultimate limit condition or showed any excessive settlement. Figure 4 shows some pile penetration records in both fill or cut ground and reveals that the foundation piles were likely to be installed to length instead of following the normal piling practice based on static calculation from the soil properties, profile and normal driving control using appropriate set criterion.

A collapse of retaining structure was also observed at the site. The retaining structure was about 3.5 m high pile supported rubble wall to retain earth at the backyard of few units of apartment structures on piles. The rubble wall was supported on two rows of piles from 15m to 24m long at pile spacing of 3.7m. At the toe of rubble wall, there was a large monsoon drain of 6m wide and 3m high. Figure 5 shows the cross section of the rubble wall and the monsoon drain. Plate 8 shows the picture of the completely collapsed rubble wall and the RC drain section. Most of the platform of these units was constructed over the previous natural stream by earth filling. The maximum thickness of fill was about 25m. The failure occurred during the monsoon month with frequent heavy raining. The preliminary finding on the cause of the collapse was mainly due to the brittle behaviour of the rubble wall when subjecting to differential distortion along the wall as a result of fill settlement. The slab supporting the rubble wall was believed to be initially designed as a seating plinth for the rubble wall, but the piles was subsequently provided with the objectives of avoiding settlement and to provide a better support to the rubble wall.

However, due to the brittle behaviour of the rubble wall, the wall cracked even at very small distortion between the piles and the unsupported mid span of the plinth after the platform settled. Once the wall cracked, the stones initially binded by the cement mortar became loose mass of stones and collapsed. Plate 9 shows the cracks of rubble wall due to such distortion. The quality of the cement mortar binding the stones was found to be very poor further aggravated the situation. Some even do not have cement mortar to bind the stones as an integrated mass. This was evident during the
detailed inspection of the collapsed rubble debris as shown in Plate 10. It was also found that the sliding resistance of the rubble wall sitting on the RC plinth was inadequate, because the timber piles were very weak in taking lateral loading, especially after the plinth was detached from the settling fill.

The collapsed mass piled up and surcharged the RC monsoon drain sections causing subsequent bending failure at the wall-base connection of the section. The detailed analyses indicate that the monsoon drain had sufficient resisting capacity to retain the backfill if the rubble wall was intact. The collapse had triggered retrogressive instability failure of the retained platform to the units of adjacent houses. However, there were no obvious distresses yet on the building.

Differential settlement of filled platform could also cause distresses to the infrastructure works. Plate 11 shows the shear cracks with the whitish calcite deposits on a cast-in-situ reinforced concrete (RC) monsoon drain due to differential fill settlement. Figure 6 shows the schematic diagram explaining the cause of the shear cracks in association with the uneven fill settlement.
There was another type of failure associated with water in this case history. Sometimes, the original topography has significant influence on the changes of groundwater regime after the topography has been changed by engineering works. For instance, filling over the previous natural stream would not necessarily stop the groundwater flow coming out in natural ways. In fact, the groundwater could rise or perch to higher level when obstructed by the earthfill. This could cause the collapsed settlement of the new fill and also unexpected water pressure build-up behind wall and the monsoon drain causing structural collapse of the wall. Plate 12 shows the structural distress of the monsoon drain suffering from excessive water pressure built up behind the wall as the weep holes did not drain off water timely.

In certain conditions, provision of weep holes at the monsoon drain sections may not function in the intended way. During heavy downpour, the weep holes could serve as the entry points for the rising swift drain water to seep into the retained soil. Subsequently, internal erosion of soil behind the drain wall could occur if the filters were not properly designed and constructed. Plate 13 shows a segmental RC monsoon drain with watermark higher than the weep holes indicating the water level behind the drain was still higher than the weep holes that did not function properly. Plate 14 shows the serious erosion at the T-junction where the cascading drain met the monsoon drain as a result of drain water ingress from the perpendicular cascading drain to the main drain. Figures 7 & 8 show the schematic diagrams on the erosion mechanism of the ingress drain water to the monsoon drain.
Sometimes, small mistake in drainage construction can cause detrimental damage to the platform formed. Plate 15 shows that the formworks were left inside the platform for an in-situ cascading drain. However, the surface runoff would choose the least resistant way and flow under gravity. As a result, the surface runoff was never drained into the in-situ drain, but ran along both sides of the in-situ drain. This was because the voids between the formwork provided a very good drainage path for the surface water. The drain latter suffered serious scouring, undermining and eventually collapsed.

Plate 15 Undermining Erosion of Cascading Drains

2.3 Case Three : Distortion of Building

This case history was featured with excessive building distortion due to differential settlement of fill of varying thickness. The two-storey terrace structures are partly located on fill and partly on cut. Due to the uneven settlement of the platform across the longitudinal axis of the terrace structures, diagonal tension cracks were observed at the structural frame. A φ150mm asbestos concrete water pipe was also damaged and leaked due to the differential settlement. The leaking water subsequently saturated the fill material and initiated collapse settlement and lateral creep movement at the side slopes. Figure 9 shows the schematic diagram of the failure and the failed slopes is shown in Plate 16.

Plate 16 Failed Slope due to Differential Fill Settlement

3 ROLES OF INVOLVED PARTIES

From the authors’ experience, the common mistakes by involved parties in most failures can be summarized as follows:

Client/Developer:

1. Fails to clearly specify end user requirements in the design brief of the consultancy services. In many cases, the client heavily relies on the consultant’s advice or even leaves it to the consultant.
2. Fails to take professional advice when making decision and design coordination among the consultants of different disciplines.
3. Does not provide adequate subsurface investigation for feasibility study or preliminary design.
4. Does not allocate sufficient budget for site supervision, sometimes remove the supervising staff strength as required for the project or take over the supervising role from the consultant, to cut down overhead, without understanding the requirements of supervision and design.
5. Having inappropriate constraints or requirements to the consultants or the contractors, such as using the unit rate for earth moving as the rate for engineered fill.
Consultant:

1. Inadequate engineering assessments for engineering design, such as evaluation of long-term settlement and fill compression problems, long-term slope stability at cut ground, negative downdrag on piles at filled ground and request for necessary subsurface investigation.

2. Mistakes or errors in the design without thorough checking and reviewing processes.

3. Improper engineering specifications, which are not specifically tailored for the project.

4. Fails to highlight to the client or coordinate with other design engineers, who will take over the site for subsequent engineering design, such as the performance of the platform.

5. Fails to provide professional advice to the non-professional client on their commercial decisions, which has design implications subsequently.

6. Does not seek input for specialist works, which is beyond the field of his or her expertise. The civil and structural consultants only emphasise on the structural design.

7. Lack of site supervision or using non-experienced supervising personnel.

Contractor:

1. Evidence of cheating on materials and poor workmanship was obvious.

2. Loss of controls over multi-level of subcontractors.

3. Not able to perform the works due to failure to recognise or understand the conditions of contract and the specification requirements.

4. Not experienced in handling specialist works.

5. Not serious in QA/QC of supervision.

4 LESSONS LEARNED

Lesson 1 : Proper Subsurface Investigation & Design

The infrastructure consultant appointed should carry out the subsurface investigation to obtain necessary parameters for the analysis and design of platform to ensure that it meets the client’s performance requirements, such as the stability, settlement including estimating the time and magnitude of the consolidation settlement. If the time of consolidation is too long, ground treatment should be considered. The foundation design should consider the settlement of platform to ensure the structural stability of the building. Cracks to building or other infrastructures would occur when the distortion due to differential settlement exceeds the tolerance limits of the buildings. Differential settlement is very prominent in undulating ground with varying fill thickness.

In the three case histories, the subsurface investigation did not have proper consolidation parameters for settlement analysis. Two of the cases do not even have any consolidation tests. The only available information is the results of standard penetration tests (SPT). This limited subsurface investigation would not provide adequate data for geotechnical analysis and design.

Desk study of the subsurface investigation should start with site reconnaissance study and the design requirements. Subsurface investigation should be properly planned and supervised by experienced personnel. Gue & Tan (2000a) provide some guidance notes on the subsurface investigation and its interpretation. Infrastructure consultant should consult a geotechnical consultant where need arises.

Sometimes, certain provision can be made in the design to facilitate ease of confirmation during site supervision. For instances, the use of spun piles or reinforced concrete piles with PVC pipe at the center of pile allow simple checking on piles integrity by the accessible void in the pile.

Lesson 2 : Settlement & Negative Skin Friction

Structural consultants for the above cases had wrongly assumed that the platforms are stable without long term settlement. Hence, the pile foundation had ignored negative skin friction (downdrag) on piles, which subsequently reduce the factor of safety of foundation when the ground settled with time. The foundation design only considered bearing capacity and some piles even terminated within the fill. Some engineers even try to ignore this reality despite the subject is part of their standard soil mechanics course. Many engineers tend to use their casual observation on the performance of other completed projects on filled platform for their judgement. However, very often, no obvious distresses were observed within the short period after the completion. This can be best illustrated with the analogy of pile load test. Just before the imposed load reaches the ultimate
capacity, the settlement of the test pile can be well within the allowable limit. However, further loading the test pile to the ultimate capacity can cause settlement of catastrophic magnitude. In other words, negative skin friction reduces the designed factor of safety on the foundation to unacceptable extent due to the designer unaware of the consequences.

Gue et al. (1999) has presented measurement of negative skin friction on pile for a bridge abutment. The results of instrumentation indicate the quantum of negative skin friction is agreeable with the estimate using the conventional neutral plane concept. It is worth to take note that negative skin friction operates at ultimate friction on the pile/soil interface with very small downward movement of the subsoil surrounding the pile. Normal magnitude of settlement in most earthwork, even well compacted, easily exceeds the critical movement for generating negative downdrag. Therefore, the quantum of negative skin friction is large and should not be overlooked. The negative skin friction can be easily revealed by instrumentation to convince the engineers and the developer.

It is also important to orientate the building layout to minimise the differential fill settlement. This can be achieved by arranging the longitudinal axis of the building parallel to the contour lines of the original topography, in which the building is underlain by fill of uniform thickness and, therefore, less differential settlement. When the relocation of the building is not feasible, slip coating or surcharging the subsoil can be used to reduce or eliminate the negative downdrag.

Lesson 3: Earthwork Compaction Control

Settlement of a filled platform is a combination of settlement of the subsoil due to the weight of the fill and the compression within the fill. The long-term settlement of subsoil and its duration are directly related to the thickness of compressible subsoil, their properties, permeability, thickness and weight of fill. The compression within fill is directly related to the degree of compaction of the fill. The settlements of the platform for all the three case histories are the results of the above combinations. Two of the case histories have hard soil lumps and boulders within the fill, which hampered pile installation giving premature pile set.

Earthwork compaction is very important, particularly within building footprint. Failure to ensure proper compaction even on incompressible subsoil would result in long-term settlement. When reaching the final platform level, strict compaction control is required to ensure formation of relative impermeable capping layer to minimise infiltration of surface water, which can trigger collapse settlement and internal erosion of the fill. This practice is also advisable at the end of the day works to prevent fill saturation by overnight rain during the progress of earthwork.

Although the degree of fill compaction can be easily checked using in-situ tests, such as JKR probe for shallow, piezocone and undisturbed sampling from boreholes for laboratory tests, it should be compacted with proper QA/QC and the design consultant team shall supervise the work to ensure proper execution of works in accordance to the specification and verification with necessary control tests. The loose thickness per compaction lift should be strictly controlled and usually should no exceed 300mm. Figure 10 shows the differences between controlled compaction (proper loose thickness) and uncontrolled compaction (improper loose thickness).

Lesson 4: Drainage Details

Overall drainage designs in many infra-structure works were not properly designed. This was mainly due to inadequate hydrography study of the original terrain and alternation of original drainage system after earthworks construction. As in monsoon drain, the engineer designed weep holes to reduce the water pressure behind the wall, but had overlooked the possible ingress of rising drain water saturating the fill during the extreme weather, which can cause collapsed compression and internal erosion of the fill. Therefore, at fill platform with potential of collapse compression, it is more appropriate to locate the weep holes above the highest design water level and design the monsoon drain section to take the full water pressure up to the weep holes behind the section during dry season.

Segmental RC drain section is not recommended for drain on fill platform because the dislodging of the segments due to differential settlement can cause serious leaking of drain water to saturate the fill and may also induce undermining of flowing water beneath the segments.

Energy breakers should be used to dissipate kinematic energy of drainage with drop fall. Sometimes, cascading drain may not be adequate to
dissipate high water flow during extreme weather condition.

To prevent soil erosion at both sides of the surface drains, provision of suitably reinforced concrete apron at two sides of the in-situ drain with thin and lightly compacted soil cover on the apron can be used to mitigate the problem. The details of the concrete apron for the surface drain as proposed by Gue & Tan (2000b) are shown in Figures 11 & 12.

Figure 10  Compaction Control

Figure 11  Typical Details of Cast-In-Situ Berm Drain

Lesson 5 : Site Supervision

Supervision is a very important factor to ensure the construction works are carried out in accordance to the design specifications and drawings. It is also crucial that the supervising personnel must have reasonable understanding of the design. Regular feedbacks of construction problems to the design office are necessary and useful for necessary design modification to suit the site condition. It will be
more appropriate to assign the supervision duty to
the design consultant as communication of matters
on supervision is more efficient within the same
office and, contractually, the professional liability is
straight forward. For large project, a resident engineer
should be engaged to supervise the works and
directly report to the design consultant. This can
also expedite the clarification process on
discrepancies on drawings, bill of quantities and
specifications between the contractor and the design
consultant.

Lesson 6: Instrumentation

Instrumentation is often missed out in many
earthworks design. It may be in the construction
drawings, but not implemented by the contractor.
This is very important for design verification and to
confirm the performance of the design. In most
cases, the simple and cost effective method of using
rod settlement gauges on the original ground surface
and surface settlement markers on the finished
platform to monitor performance of earthwork
platform. With these instruments, the settlement of
subsoil and the fill compression can be recorded.
For thicker fill, multilevel extensometer can be used
to record the relative settlement at respective levels
to a reference datum and provide the settlement
profile of the compressible layer. Settlement profile
will be useful to determine the neutral plane for
estimating negative skin friction. In the foundation
design with the problem of negative skin friction,
design verification by instrumentation and necessary
testing are required as the uncertainty in estimation
of the downdrag force is much higher. Multi-level
strain gauges installed at the test piles with
calibration during pile load test can be used to study
the load transfer mechanism of the pile/soil interface
for estimating negative skin friction and for long-
term downdrag monitoring.

5 PREVENTION OF FAILURE

Before preliminary design stage, the Client should
identify the scope of respective consultants to their
expertise, preferably with the input from project
management specialist. It is also important that
experienced personnel in the consultant firm to be
involved in the consultancy service rendered.
Organisation chart and the curriculum vitae of the
involved personnel for the project should be
submitted to the client for review and approval
before the engagement.

If specialist input is required, which is very
common in most civil engineering projects, there
shall be no hesitation to get specialist to involve in
the development of the preliminary design and
subsequently in the detailed design. Very often, the
specialist input is only requested after the project
encounters problems. Many geotechnical failures
will eventually lead to serious structural failures as
the structural design has no considerations for the
adverse consideration of geotechnical aspects. The
consequential costs of remedial works are expensive
in addition to the delay in delivering the project. In
fact, appointment of specialist would enhance the
safety and value engineering of a project.

In all design process, there should be a systematic
way to check and review the designs. The checking
is to eliminate potential mistakes in the design input,
calculations and the design output, whereas design
review is a process of ensuring the adopted
methodology and design concept are in good order.
The design procedures and checklists to the ISO
requirements are very useful for the checking and
reviewing a design in a consistent manner. Ling
(1998) has given a very elaborative discussion on
design review in accordance to ISO 9001 Quality
System Requirements.

For large project, it is always advantageous to
have an independent consulting engineer to review
the consultant’s design to prevent serious design
errors and possibly suggest some design
improvements for value engineering and also,
possibly, savings in terms of construction time and
costs.

For all project, especially if a contract is on
performance basis or end product basis, QA/QC
procedures by the contractor to ensure construction
control to meet the specified performance
requirements should be submitted to the consultant
for review and approval. This should be the
commitment from the contractors to officially
substantiate his attempts to achieve quality works
with the available resources and also to provide a
good official reference for the construction
monitoring of the design consultant.

Lastly, site supervision by the consultant should
not be compromised. The level of supervision
should be determined based on the scope of works.
The size of the supervising team depends on the
scope, extent and nature of works, and time period of
the contract. Usually, a team of supervising
personnel is required for construction works of
different engineering disciplines or concurrent sessions of different construction activities. The developer should also insist full time site supervision and frequent site inspection or visit by the design team to ensure the compliance of works to the design and specifications.

The consultant should also fully understand and comply with the laws (Sections 5 & 7 of Uniform Building By-Laws) and the statutory requirements on site supervision (Form E for the local authority). Site supervision is further highlighted in a circular from Board of Engineer, Malaysia (BEM, 1989) to all registered engineers.

6 CONCLUSIONS

The three case histories summarise and highlight the following important attributes for a successful housing project.

All filled platforms should be properly designed with specialist input where required and constructed with adequate QA/QC procedures.

Supervision by design consultant is a must to ensure compliance to design requirements, especially removal of topsoil, benching and compaction for any earthwork project.

The magnitude and duration of settlement of subsoil and fill compression should be predicted and verified by settlement monitoring.

Settlement of platform also reduces bearing capacity of end bearing piles due to generation of negative skin friction on piles.

7 REFERENCES

