Geotechnical Instrumentation Monitoring: A Transition from 3A to 5A Approach

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ABSTRACT

The referenced project is a cut-and-cover tunnel constructed in a dense urban area with a long history of reclamation and aged buildings. A critical section encroached into a historic two-block premises that require enhanced protection. Block 1 is a reinforced-concrete frame on cast in-situ piles, while Block 2 is a brickwork structure on shallow footings. Although PNAP APP-18 and APP-137 prescribe traditional 3A monitoring levels, these were considered too rigid for a project of this scale and complexity. The project team therefore introduced a revised third A level before the second A level was triggered, using updated settlement predictions and mitigation measures to maintain control. PLAXIS was used to model pile–soil interaction and verify geotechnical safety factors, while SAP2000 assessed structural integrity under predicted movements. Regular instrumentation meetings ensured that monitoring data were reviewed alongside construction progress, allowing timely refinement of mitigation strategies and further adjustments to the revised third A levels. This adaptive, performance-driven approach anticipated the flexibility later formalized in the 5A framework of PNAP APP-137, demonstrating that large projects can balance efficiency with robust engineering safety.

Keywords: Damage Classifications of Buildings, Deformation Modes, Limiting Tensile Strains

1 INTRODUCTION

In Hong Kong, statutory regulations require the submission of a monitoring plan when construction works may affect the nearby buildings, structures, land, streets, or utilities (BD 2004). Traditionally, monitoring checkpoints and instruments are evaluated against three triggering levels, known as the Alert Alarm Action or 3A system, which dictate the response actions approved by the Buildings Department (BD) and Geotechnical Engineering Office (GEO) or supervising officer. These levels are determined based on the site conditions, nature and proximity of sensitive receivers to the construction site. Before 2012, there is no guideline on specific response levels for the 3A monitoring system except the third A level for utilities, which required agreement with the relevant government department or utility provider to reflect the maximum tolerable movement for the service in question (BD 2004). Since these 3A values were not directly correlated with the predicted settlements, making their use as control levels appeared potentially misleading.

A significant change occurred in March 2011 when the West Kowloon Terminus (WKT) of the Express Rail Link project adopted an innovative instrumentation monitoring plan (So et al. 2013), which is inspired by the Generalized Plan of Action and the Detailed Plan of Action outlined in the MTR Standard Material and Workmanship Specification (MTR 2009) and Project Integrated Management System (MTR 2010). In February 2012, the BD formalized detailed monitoring requirements in the Practice Notes for Authorized Persons (PNAP) APP-18 (BD 2012a) and APP-137 (BD 2012b), closely resembling the WKT approach. The 3A levels were explicitly defined as Alert, Alarm, and Action. PNAP APP-18 (BD 2012a) outlined the reference contingency measures for piling works at each level, to be implemented by the Registered Structural Engineer (RSE) and Registered Specialist Contractor (RSC). Meanwhile, Appendix B in the PNAP APP-137 (BD 2012b) recommended ground settlement limits for piling and related operations, along with the reference 3A values for ground settlement markers, utility settlement markers and building tilt markers, based on serviceability criteria. As different structures can exhibit varying tolerances to their movement and foundation displacement, the acceptability of estimated ground settlements must be assessed case by case, considering their structural integrity, stability, and functionality.



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While the 3A values and monitoring requirements in the PNAP APP-18 (BD 2012a) and APP-137 (BD 2012b) are generally suitable for small-scale projects, they present limitations for larger, more complex developments. An example is the construction of a cut-and-cover tunnel underneath two operating flyovers in old reclaimed urban land surrounded by old buildings on piles or footings, and at a location partially encroached into a historic building premises. It pioneered a revision of the Action Level (third A) before the Alarm Level (second A) was triggered after in-depth building condition surveys, additional ground investigation, interpretation of monitoring data, implementation of mitigation measures, more rigorous design reviews and settlement predictions. Further adjustments to the mitigation strategies and revised Action Levels were implemented as needed.

In November 2024, PNAP APP-137 (BD 2024) was updated to introduce a refined 5A system, closely resembling this project's approach. This revision retains the Alert and Alarm Levels but subdivides the Action Level into Action Levels 1, 2 and 3. Unless sensitive structures or utilities are nearby, empirical limits provided may serve as provisional trigger values for pile foundations and ELS works. The Technical Guidance Note No. 54 (TGN 54) by GEO (2024) also provides similar guidelines on ground deformation control mechanism for geotechnical works. This enhanced framework aims to improve the risk management for large-scale projects while safeguarding surrounding infrastructure.

Thus, this project may serve as a case study for its pioneering and resemblance to the 5A system. It highlights critical factors such as site-specific constraints, geotechnical challenges, design rationale, preventive measures, preemptive mitigation strategies, construction execution, and with main focus on the engineering approach to revise the 3A triggering criteria to protect the surrounding buildings, in parallel with the project advancement.

2 DEFORMATIONS AND DAMAGE TO BUILDINGS ADJACENT TO EXCAVATIONS

2.1 Deformation Modes and Damage Classification

Literature about damage due to deformations usually relates to the curvature of building because more curvature means higher strains and more damage. Curvature K can be calculated as a derivative of deformation, and radius $R = 1/K$ is a similar indicator for possible damage. Curvature of a building can arise from various modes of deformation such as shear deformation, bending, extension, compression, and a combination of deformation modes generally occuring simultaneously. When evaluating potential damage to buildings, it is crucial to distinguish various deformation modes associated with the damage (Bonshor and Bonshor 1996). Most damage occurs from cracks, so that crack degree is the main indicator of damage to a building. However, they do not necessarily indicate the extent of damage if stand alone. Table 1 shows two common classification systems. Skempton and MacDonald (1956) classified the amount of damage based on deformation into three broad categories, while Burland et al. (1977) refined these broad categories based on cracks into six categories, numbered 0 to 5 with increasing severity based on the studies by Skempton and MacDonald (1956), Grant, Christian and Vanmarcke (1974). These data sets apply to what have been traditional types of structures in the past, mainly buildings with frames and block or brick walls, or older buildings. No further updated state-of-the art paper is published since then. Notwithstanding this, both systems are widely used for building damage assessment due to deep excavations and other causes. e.g. BRE (1981) and the Institution of Structural Engineers (1994).

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Table 1: Damage Classification System for Buildings

Damage Description	Limiting Relative Rotation
Structural Damage	1/150
Cracking in Walls and Partitions	1/300
Safe Limit against Cracking	1/500

(a) Skempton and MacDonald (1956)

Category of Damage	Degree of Damage	Description of Typical Damage	Approximate Crack Width (mm)	Limiting Tensile Strain (%)
0	Negligible	Hairline cracks	< 0.1	< 0.05
1	Very Slight	Fine cracks easily treated during normal redecoration. Perhaps isolated slight fracture in building. Cracks in exterior visible upon close inspection.	0.1 – 1.0	0.05 – 0.075
2	Slight	Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible. Some repainting may be required for weather-tightness. Doors and windows may stick slightly.	1.0 – 5.0	0.075 – 0.15
3	Moderate	Cracks may require cutting out and patching. Recurrent cracks can be masked by suitable linings. Tuck pointing and possible replacement of a small amount of exterior brickwork may be required. Doors and windows sticking. Utility services may be interrupted. Weather tightness often impaired.	5.0 – 15.0 Several closely spaced cracks > 3.0	0.15 – 0.3
4	Severe	Extensive repair required involving removal and replacement of walls especially over doors and windows. Window and door frames distorted. Floor slopes noticeably. Walls lean or bulge noticeably. Some loss of bearing in beams. Utility services disrupted.	15.0 – 25.0 Depending on the number of cracks	> 0.3
5	Very Severe	Major repair required involving partial or complete reconstruction. Beams lose bearing. Walls lean badly and require shoring. Windows broken by distortion. Danger of instability.	> 25.0 Depending on the number of cracks	> 0.3

(b) Burland's (1977)

2.2 Deformation-Based Approach based on Empirical Criteria for Damage

The limits for damage to buildings are available in various forms and can be derived either theoretically or from field observations. The simplest limits are in the form of maximum settlement of the structure or differential settlement; while others describe the maximum rotation of the building. More advanced damage assessment approaches consider the deflection ratio or relative rotation. The current state of the art for predicting building damage from expected deformations is to calculate the strains from deflection ratio or relative rotation coupled with finite element methods. Table 2 is a summary of empirical criteria of limiting deformations for frame building and unreinforced load bearing walls. Meyerhof (1953) estimated the total and differential settlements of foundations in relation to the soil and foundation characteristics and the structure stiffness. The settlement stresses in foundation and superstructure are considered, and the recent observations on brick walls and cladded building frames are analysed. Skempton and MacDonald (1956) are the first to describe the effect of building damage. They introduced the criterion Δ/L with Δ and L being the differential settlement and distance between two footings or points on a building, which was later defined by Burland (1977) as relative rotation ß. Based on observations and theoretical study, Polshin and Tokar (1957) used 0.05% as the limiting tensile strain for brick buildings with $L/H > 3$ where L is the distance between two separating joints which is not the necessarily the whole building length, and H is the height of the building. Boscardin and Cording (1989) combined Δ/L and horizontal strain into a damage matrix of which tensile strain > 0.05% implies risk of cracking while $\Delta/L >$ 1/300 implies structural damage.

Table 2: Empirical Criteria for Damage (After Korff 2009)

Damage Description	Limiting Relative Rotation □	Source
Structural Damage	1/150	Skempton and MacDonald (1956)
Cracking in Walls and Partitions	1/300	
Safe Limit against Cracking	1/500	
Cracking to No Infill Structures or No Danger of Damage to Cladding	1/200	Polshin and Tokar (1957)
Cracking to Steel and Concrete Frame Infilled Structures	1/500	
Cracking in Open Frames	1/300	Meyerhof (1953)
Cracking in Infilled Frames	1/1000	
Cracking in Load Bearing walls or Continuous Brick Cladding	1/2000	

(a) Limiting Deformations for Frame Buildings

Damage Description	Limiting □/L	Source
Sagging $L/H \leq 3$	1/3300 – 1/2500	Polshin and Tokar (1957)
Sagging $L/H \geq 5$	1/2000 – 1/1400	
Sagging	1/2500	Meyerhof (1953)

(b) Limiting Deformations for Unreinforced Load Bearing Walls

2.3 Crack-Based Approach based on Limiting Tensile Strains

Burland and Wroth (1974) first introduced limiting tensile strain to relate the effect of ground displacement into the strain in a building. They assumed the building as a simple beam such that when settlements affect the building, direct tensile strains occur due to bending deformation and diagonal strains due to shear deformation. The load can be modelled by a central point load or a uniform load. The neutral axis for bending is assumed to be at the bottom of the beam or wall in hogging and at mid-sp

an in sagging. Boscardin and Cording (1989) expanded this model to include horizontal strains to the bending and shear deformations based on angular distortion because deformations caused by excavation and tunnelling cause horizontal strains as well. Mair et al. (1996) and Burland et al. (2004) further refined this by integrating Mohr's circle for combined bending and shear strains. Son and Cording (2005) improved their previous works and got a lateral strain independent of L/H, E/G and the position of neutral axis, where E is the elastic modulus and G is the shear modulus of the building. However, e.g. CIRIA (2003) reported that low rigidity buildings like brick walls are probably governed more in bending than shear. In cases where large openings exist, buildings will behave more flexible and shear might govern instead. Boone (2001) took a similar approach but assumed a uniform loaded beam with neutral axis at mid-span. He considered critical tensile strains to be the absolute minimum of 0%, noting pre-existing cracks may widen under strain.

2.4 Drawbacks of Deformation and Crack Based Approaches

Despite these two approaches being widely used, they have some drawbacks. The assumption of isotropic building behaviour is often unrealistic because real structures exhibit orthotropic properties, with varying ratios of bending to shear stiffness (E/G) (Burland and Wroth 1974). The distribution of shear strain over building height and long buildings is another concern because Netzel (2005) found that shear strain varies over height and ignoring unaffected sections in long building may lead to non-conservative strain predictions. The use of deflection ratio Δ/L or relative rotation β is contentious. CIRIA (2003) considered that Δ/L is better to describe bending strains and β is more suitable for shear deformation, while a building that bends rather than rotates usually experiences more damage. Netzel (2005) also demonstrated inconsistencies in strain estimation when applying these methods to real analysis, complicating their interchangeability. Finally, rigid body tilt introduces ambiguity. Although tilt is often excluded from strain calculations, it can indirectly induce damage through gravity-induced stresses, particularly in framed structures with isolated footings. Modern approaches will use finite element to simulate soil-structure interaction to predict deformation-induced damage and real time monitoring to provide building deformation data and surface crack movement. Notwithstanding these drawbacks, the tensile strain methods can provide a valuable framework to assess potential damage to building due to construction activities and is also adopted in this project.

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3 SITE GEOLOGY

The said project is located on a reclaimed land with a complex history of reclamation and use over the past 2 centuries. As shown in Figure 1, the existing ground level is generally flat at +4mPD and groundwater level generally fluctuates from +0.5mPD to +2.8mPD. The ground comprises of reclamation Fill overlying Marine Deposits (MDs), Alluvium, Residual Soils and Granitic bedrock. Since reclamation in the old days is by end-tipping from trucks without dredging and is uncontrolled, the Fill can be very loose without compaction and is expected to be placed on in-situ large sea wall stones, cobbles, boulders, timber, dead vegetation, abandoned drainage pipes and debris which were not removed prior to reclamation. This will give rise to a fill layer ranging from a few meters to more than 10m with large variability in properties and conductivity. The MDs are soft to firm silty clay with inter-bedded deposits of Marine Sand. The reclamation can also give rise to disturbance to the MDs in the form of mud waves leading to locations of very soft disturbed MD which were apparent. The undrained strengths and thicknesses of the MDs were highly variable. The Alluvium is firm or dense but locally soft or very soft. The interface between the MDs and underlying Alluvium is irregular. Further below is varying thickness of completely to highly decomposed Granite and then bedrock with locally abrupt change of rockhead level ranging from -15mPD in the east to -60mPD in the west.

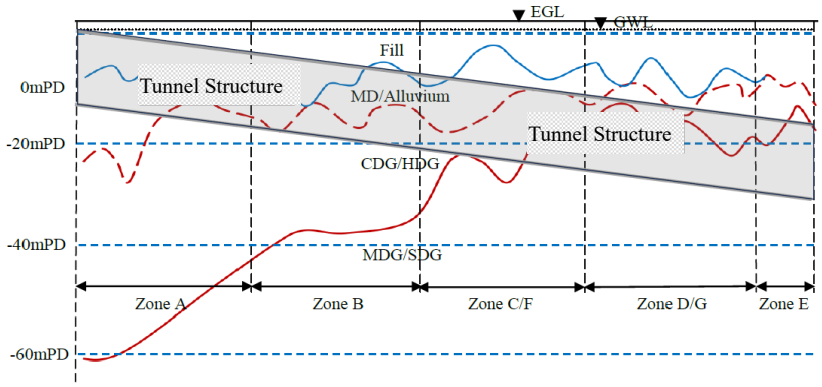


Figure 1: Geological Stratigraphy of the Site

4 DESIGN AND CONSTRUCTION

4.1 Original Engineer Design and Construction

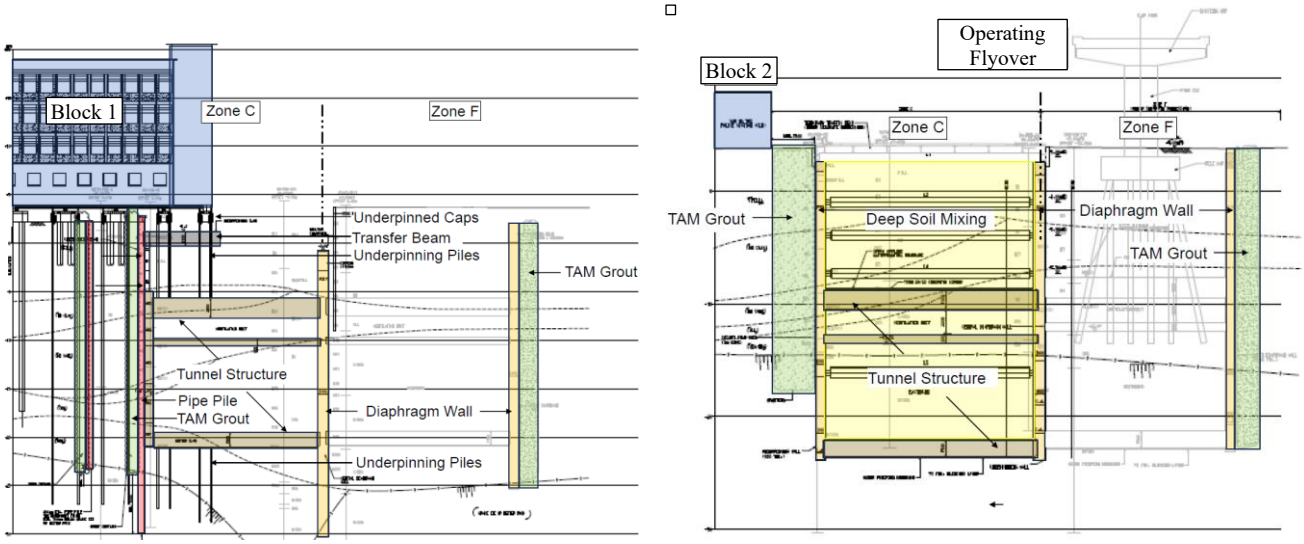
The conforming excavation and lateral support (ELS) system is very robust in stability and hydraulic cut-off, adopting diaphragm wall panels as the vertical elements. These wall panels would penetrate sufficiently into bed rock. Top-down construction was adopted to minimize the impact to the surrounding environment and existing traffic. Basic construction sequence involved installation of diaphragm wall which serves as the permanent tunnel wall, construction of a temporary steel deck for temporary traffic diversion, excavation and lateral support, and construction of tunnel slabs as the excavation progressed in a top-down construction approach. The majority of the tunnel construction was carried out underneath existing flyover with continuous live traffic maintained, resulting in a significant headroom constraint.

4.2 Alternative Contractor Design and Construction – Challenges and Measures

Alternative designs were proposed to reduce rock excavation by trench cutter. Proposed was an underpinning wall in Zone E, temporary pipe pile wall embedded into rock with in-situ concrete wall for the part encroached into the historic premises and in high rock head locations. However, the alternative pipe pile wall design was subjected to risk of seepage at the wall openings that enabled utility crossings, the interface between the diaphragm wall panel and pipe pile, gaps between the pipe piles, at the pile toes and the rock joints if the rock quality is poor.

Figure 2 shows the alternative ELS design adjacent to the historic premises. As part of building Block 1 encroached into the tunnel alignment, a pipe pile wall with curtain grout was used instead of the diaphragm wall panels to avoid clash with the existing building foundation, and affected foundation piles were also removed.

The ground floor slab was removed, and pipe piles were installed under low head room conditions, limited by the ceiling of upper floor slab. Tube-a-manchette (TAM) grouting was designed between and at the back of pipe piles to stop seepage through the wall, which were strengthened with steel plankings as excavation progressed. Fissure grouting was proposed in rock with poor quality to stop seepage from the bottom of the excavation. Prior to bulk excavation works, preventive measures were also implemented. Polyurethane grout was injected through the holes drilled in the floor slab to stabilize the underlying ground and fill any potential voids to prevent building settlement. Temporary micro-piles with stitching to the existing building caps in the area within the tunnel footprint were first installed, then ground improvement was implemented using TAM grouting underneath footprint area of caps, to reinforce the building foundation and to prevent building settlement. Also, 406mm diameter socketed H-piles were proposed under low head room condition (less than 4m). Upon partial transfer of the existing building load to the new foundation system, the loading from the existing piles and associated temporary micro-piles could be neglected during tunnel excavation.



(a) Section across Block 1 (b) Section across Block 2

Figure 2 – Alternative ELS Design adjacent to the Historic Building

5 MONITORING AND RESPONSE ACTIONS – A CONVENTIONAL 3A APPROACH

Major diaphragm wall construction was started in mid-2018, followed by bulk excavation in phases. As the works progressed, the instrument readings were measured and monitoring data was made available online daily. Should there be any exceedance of the 3A values, a short message service (SMS) together with an email would be sent to the relevant persons. In addition to the statutory and contractual requirements, regular instrumentation monitoring meetings were set up. Instrument readings were reviewed and collated with construction activities. If the measured data exceeded any of the A levels, immediate review would be carried out and mitigation actions would be discussed and confirmed to implement in the meeting in accordance with the instrumentation monitoring system and reporting procedures established under the contract. Specific task force on this aspect had also been established during the critical stages to oversee the process. Minute as a record, and formal submissions were followed up as necessary. This collaboration with mutual respect enabled clarity of works and acquired practical and prompt decisions, avoiding communication in writing and dragging on submissions.

5.1 Excessive Drawdowns, Investigations and Remedial Measures

Initial settlements were generally within design predictions. However, progressive increase in settlements over a large area were observed after successful completion of pumping test and start of excavation in a zone. Thorough investigation found that the causes of increased settlement were due to slight water ingress through the cofferdam and bursting of a watermain at distant. Following the action plan in the alternative pipe pile wall design, preempt remedial measures were implemented. Review was immediately carried out and tube-a-manchette (TAM) grouting enhancement measures was agreed as an appropriate scheme. TAM grouting was

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carried out using inclined holes from ground along the two blocks. Monitoring records showed that the ground water tables outside the cofferdam restored and the settlements were stabilized. Site inspection found that the excavation bottom was generally dry, and this indicated that the remedial grouting below excavated soil was effective. The effectiveness of remedial grouting was then verified by another pumping test without carrying out recovery phase to avoid flooding of the excavation, which was confirmed by a joint-inspection after submission of the pumping retest assessment report.

6 DESIGN REVIEW AND THIRD A REVISION – TRANSITION FROM 3A TO '5'A APPROACH

Besides the above, a task force involving some geotechnical experts was set up to conduct a holistic review and establish actions in controlling the settlement issue. In order to avoid any unrecoverable adverse effect to the concerned structures and potential interruption or suspension of works when the third A is reached, the allowable water drawdown and predicted building settlements were reviewed incorporating new observations and the triggering values were revised.

6.1 Resetting of Baseline Readings and Revision of Third A Values – A Step from 3A to '5'A

The instruments for the said project were installed as early as mid-2018. Since then, the site conditions had largely been changed due to different land use, site construction and traffic diversions. As such, a holistic review was conducted in December 2022. All unnecessary instruments were deleted and at some locations, where required instruments were added. Since soil is a particulate material, it can change in volume when it is loaded or unloaded due to changes in effective stress. Thus, lowering groundwater level will increase soil load causing compression along the λ-line, while rising groundwater level will reduce soil load leading to swelling along the κ-line. Given that the historic low groundwater level is 0mPD to -1mPD, settlement from drawdown to this range has already occurred, so that revisiting it would not cause further settlement. Ground settlements would diminish with distance from source and were predicted for further groundwater drawdown to -3mPD following Peck (1969) given in Figure 46 in the Review of Design Methods for Excavation (GEO Publication 1/90) by GEO 1990. Among them, the revising and updating of the third A levels at the historic premises is a notable example to discuss in details.

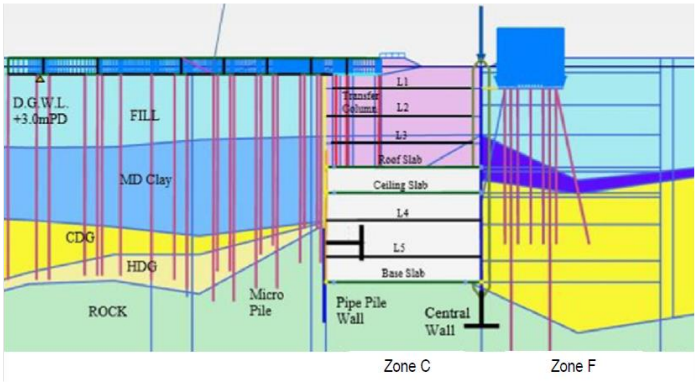
6.2 Block 1 – Reinforced Concrete Building on Cast In-situ Reinforced Concrete Piles

Block 1 is a 4-storey reinforced concrete building constructed in 1957, with a height of 14.5m and a footprint of 24.5m × 40.7m. The structural system consists of a beam-column frame with a core wall to resist lateral loads. Typical column grids are spaced at approximately 5.1m. The foundation features isolated pile caps connected by tie beams, supported by 80 number about 22m long 0.8m diameter cast-in-situ concrete piles. Appraisal was conducted to evaluate the building's structural adequacy under Ultimate Limit State (ULS) and Serviceability Limit State (SLS), incorporating the soil-structure interaction effects due to prescribed ground settlements. The assessment was focused on the structural capacity in bending and shear, pile-soil-structure interaction under differential settlement and stability against soil collapse using c'-ϕ reduction analysis.

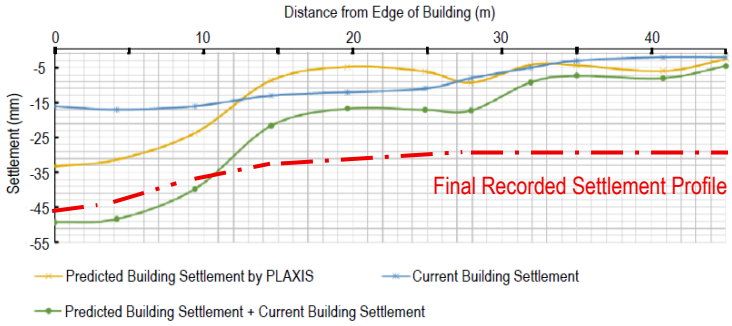
In the pile-soil-structure interaction analysis, a two dimensional PLAXIS 2D model was developed as shown in Figure 3(a) to simulate the ground movement induced by tunnel construction, accounting for the limited stiffness of the existing ELS system. A 3 dimenional PLAXIS 3D model was also used to analyze the full foundation system, including piles, with c'-ϕ reduction analysis to determine the factor of safety against soil collapse. Predicted ground settlements were exported as contour plots. The original predicted settlement profile along the building, the current settlement profile and predicted settlement profile after updating with current monitoring data and design review is shown in Figure 3(b). A value of 45mm maximum settlement was predicted at the edge of the building.

For the structural analysis, a 3 dimensional SAP2000 model was used to analyze the load effects to the space frame of columns, beams and slabs under the imposed settlements. Critical elements such as the ground-floor columns and first/second floor beams were prioritized due to their higher stiffness demands under settlement-induced loading. The building was assumed to deform as the predicted ground settlemnt profile. The differential settlement using angular distortion approach by Skempton & MacDonald (1957) in Table 1(a) was used to appraise the structure. The relative rotation or angular distortion was calculated as the ratio of differential settlement to the distance between two points, excluding the tilting effects. The computed angular distortion

was equal to 1/450, which falls within the negligible to very slight structural damage range. Under these conditions, the utilization ratios of critical elements were assessed and were determined equal to 0.82 in hogging and 0.78 in sagging for first/second floors beams and 0.85 in axial/flexural for ground floor columns. All values remain below 1.0, confirming the adequacy under ULS. Furthermore, soil-structure interaction using c'-ϕ reduction analysis demonstrated that a FOS = 17.7 is obtained across all soil strata, significantly exceeding the minimum requirement of 3.0. This shows that soil collapse is not a governing factor, which implies that the settlement limits are driven by the structural tolerances rather than geotechnical failure. On integrating the above outcomes, the third A level was revised to 45mm total settlement with 1/450 angular distortion.



(a) Modeling for Excavation adjacent to Block 1



(b) Measured and Predicted Settlements

Figure 3: Prediction of Settlement Profile of Block 1

6.3 Block 2 – Historic Brick Building on Footings

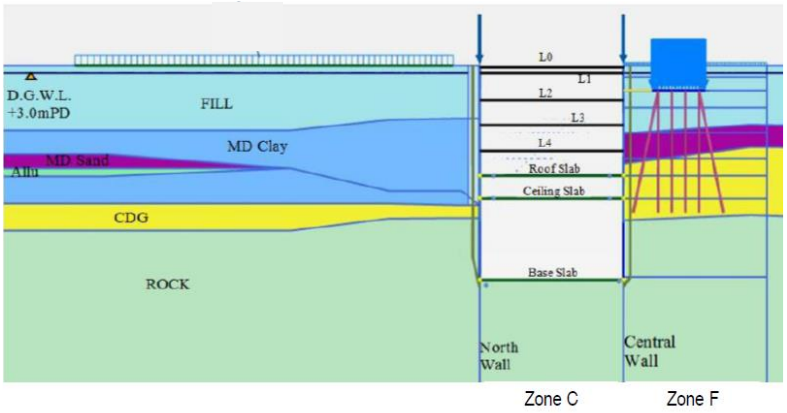
Block 2 is a historic 3-storey brick building constructed in 1922. It stands 11.69m tall to the roof slab and 15.8m to the ridge of its tiled roof. The structure consists of two blocks measuring 46.8m × 16.5m and 34.7m × 15.4m, with a reinforced concrete beam-slab system supported by load-bearing brick piers and walls. As there were no piling records available, the foundation is assumed to be strip or pad footings of 300mm thick with top level at +3.09mPD based on trial pit survey findings. Condition survey conducted in 2018 showed that the conditions of the structure was basically satisfactory. Obvious visible defects such as cracks and concrete spalling for all structural elements were not observed.

Given the ongoing deep excavations adjacent to the building, an assessment was conducted to evaluate the potential ground movement effects. The methodology combines geotechnical analysis and structural calculation based on established engineering principles, particularly those outlined by Burland (2012). Three structural sections along the building were considered. In the geotechnical analysis, a two dimensional PLAXIS 2D model as shown in Figure 4(a) was developed to simulate the greenfield ground deformations and stress changes due to excavation. Building loads were modelled as plate elements and the resulting settlement profile was derived. Of the structural assessment, the limiting tensile strain methods by Burland and Wroth (1974) and Boscardin and Cording (1989) were followed to estimate the building response. The structure was conservatively assumed to behave as a weightless elastic beam subject to full greenfield ground movements. Strains were calculated for sagging and hogging zones by analytical beam equations. Damage classification followed Burland's (1977)

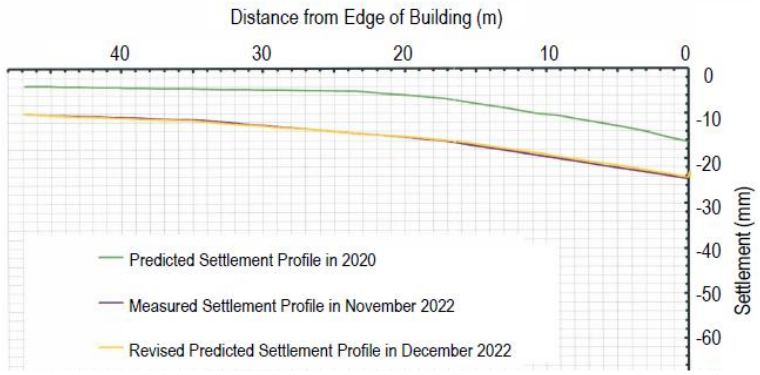
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system in Table 1(b). This is also adopted by BRE (1981) and Institution of Structural Engineer (1978 and 1994). Due to limited as-built records, design practices for 1920s were inferred from the ICE Manual of Geotechnical Engineering (Burland et al. 2012) and the Settlement of Buildings and Associated Damage by Burland and Wroth (1974). Calculated settlement profile in Figure 4(b) shows that 34mm maximum settlement occurs at the southern edge of the building and hogging deformation dominates beyond 30m from the nearest building edge.



(a) Modeling for Excavation adjacent to Block 2



(a) Measured and Predicted Settlements

Figure 4: Prediction of Settlement Profile of Block 2

Structural responses are calculated giving 0.06% maximum tensile strain. This is in Category 1 (i.e. very slight damage with cracks ≤ 1mm) with angular distortion approximated to 1/500 differential settlement which is also within the safe limits given by Skempton and MacDonald (1956) (Table 1a). Based on these predictions, the third A level was revised to the maximum tensile strain limit = 0.08% (i.e. Category 2 with slight degree of damage) and differential settlement limit = 1/500 (i.e. safe limit against cracking), with 45mm maximum settlement. With these revisions, Block 2 was expected to experience "very slight damage" (Category 1) under the predicted ground movements. Notwithstanding these findings, extensive grouting was applied to ensure settlements remained below 45mm to prevent exceeding the angular limits.

However, larger settlements than the above predictions in Block 2 were observed in 2022 after start of excavation and later bursting of a watermain at distant. Sensitivity design checks by varying the soil stiffness and ground water level along the road in the geological model to mimic the measured settlements suggested that the bursting of watermain was likely the main cause of under-prediction. Additional boreholes considered that the adopted soil stiffness is appropriate while additional standpipes and piezometers revealed that extensive water drawdown occurred along the road. This is likely due to the large conductivity of the fill and probably amplified by the damping effect after construction of the diaphragm walls and injection of extensive TAM grouting. Nevertheless, the geological model was revised with variation of ground water level extended to a larger distance to mimic the measured settlements. Further predictions showed that the differential settlement limit would remain at 1/400 with 0.09% maximum tensile strain and 64mm maximum settlement. The third A level was revised to 1/400 differential settlement limit (i.e. safe limit against cracking) with 0.09% maximum

tensile strain limit (i.e. remain in Category 2 with slight degree of damage) and 80mm maximum settlement (Figure 5). Since the drawdown effects were distant and could extend to the left periphery of the building, further TAM grouting was applied in stages to this area to control the relative settlements/angular distortions.

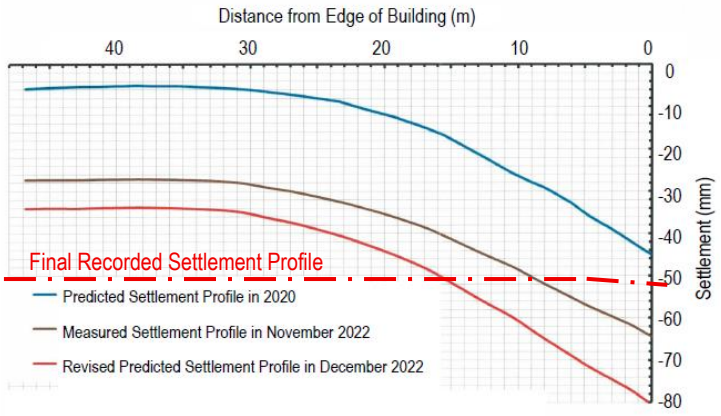
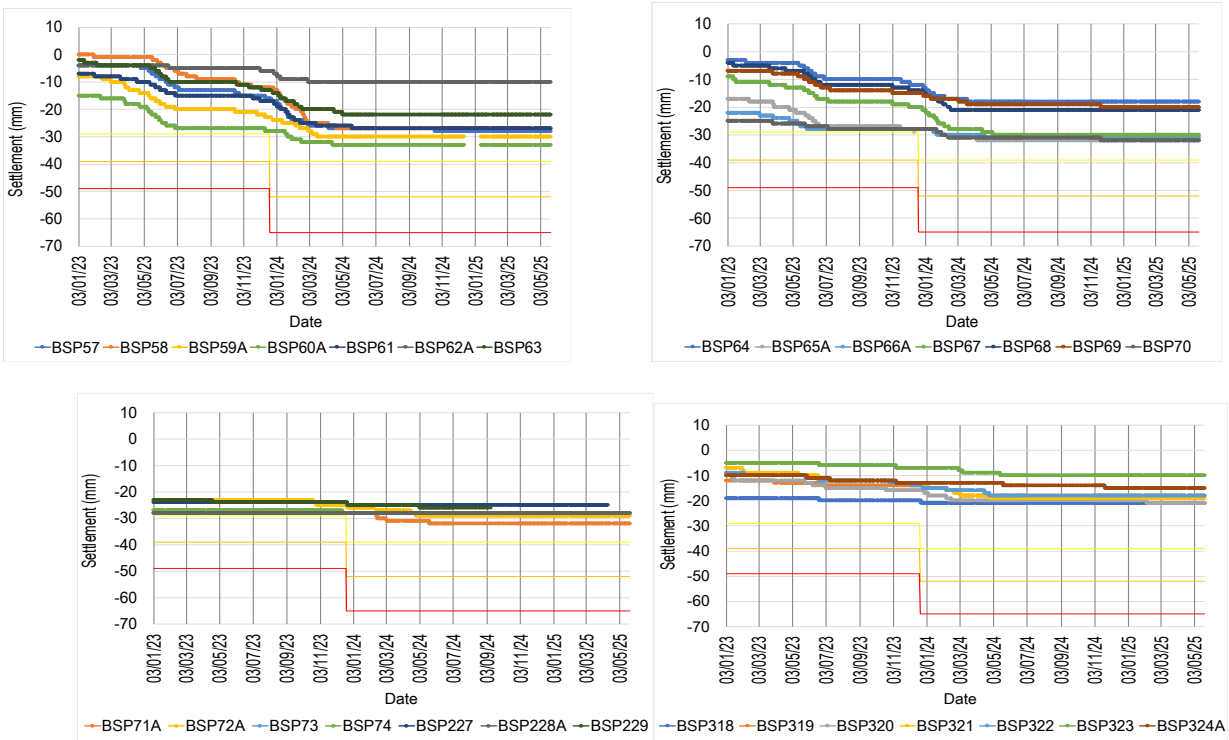


Figure 5: Revised Prediction of Settlement Profile of Block 2

7 DISCUSSIONS AND CONCLUSIONS

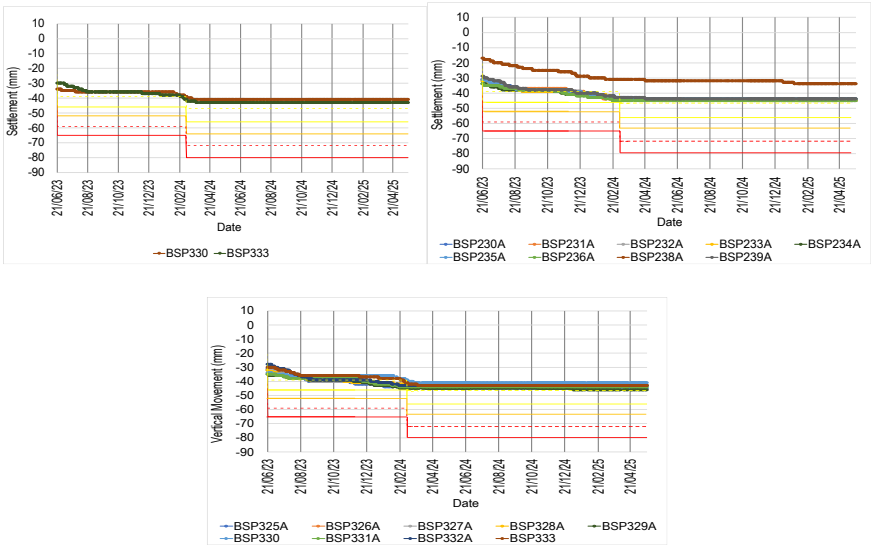
All excavations were basically completed in May 2025. Figure 6 shows the settlements of the historic premises continued to gradually increase but became stable in April/May 2024 when the base slabs started to construct. This proves that the new monitoring approach with the remedial measures and additional preventive measures are effective and appropriate. Despite the measured settlements are much smaller than the predicted maximum settlements, the settlements distant from the excavation are found much larger than the predicted settlements for Block 1 and Block 2. This is likely attributed to the long-term settlement arisen from the consolidation of underlying Marine Deposits due to fluctuation of ground water level during excavation.



(a) Measured Settlements at Block 1

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(b) Measured Settlements at Block 2

Figure 6: Measured Settlements of the Historic Premises to Completion of Excavation

Thus, the said project can serve as a notable example showing limitations of the 3A approach (BD 2012) and innovations to overcome the challenges in a large and complicated project before launching of the 5A approach (BD 2024). To avoid exceedance of the third A leading to project progress, a very robust ELS system was proposed along with additional preventive and enhancement measures such as TAM grout, polyurethane grout, DCM grout and underpinning of the existing building. It is no doubt that time-savings would be one of the benefits from this pro-active approach during design reviews and provides more suitable predictions of ground response to the satisfaction of the governing authorities. In this regard, the task force decided to execute all these actions in advance before the second A was reached, following the concept of Generalized Plan of Action by MTR (2009).

This action pioneered a new approach to review and reset the third A value based on sound engineering principles before the second or third A was reached, resembling the 5A approach promulgated by BD in 2024 (BD 2024). A holistic review such as building condition surveys, additional ground investigation, interpretations of monitoring data, implementation of mitigation measures, more rigorous design review and settlement prediction was required prior the revision of the third A. The interim trigger of the 5A approach provides an early warning and provide opportunity for early planning on preventative measures such that the critical SLS condition level does not occur. Particular focus in this paper is given to the protection of buildings, using a historic building with one block in reinforced concrete on cast in-situ piles and another block in brickwork on footings as an example. It explained how the controlling criteria, in terms of settlement, relative/differential settlement and angular distortion were established, and work approaches on how the triggering values for reinforced concrete and brickwork structures were revised based on the existing damage classification systems. Further adjustments to the mitigation strategies and revised Action Levels were implemented when needed and mastered. This enhanced framework, resembling the 5A approach and requiring collaboration of the project team, can improve the risk management for large-scale projects while safeguarding surrounding infrastructure.

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