Assessment of Foundation and Excavation and Lateral Support (ELS) Works for a Footbridge near MTR Structure in Admiralty, Hong Kong

Y. T. Shang, L. M. Wei
Ove Arup & Partners Hong Kong Ltd., Hong Kong
Y. Chen
China State Construction International Holdings Ltd., Hong Kong

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ABSTRACT

Pile installation along the perimeter of, or within, the excavation area for Excavation and Lateral Support (ELS) works are common construction practice. Both foundation and ELS works that employ piling techniques are widely accepted, but they invariably induce vibration, requiring careful measures to minimize impacts on the surroundings. This technical paper presents the impacts of piling and ELS works for a newly completed footbridge adjacent to underground structures. Special attention is given to pile drilling in soil near an existing MTR station and the vibration generated. Moreover, the sensitivity of a heavily trafficked main road and the dense urban environment and the presence of nearby buildings, structures, and utilities increase the complexity of the works. To mitigate vibration across various soil types, adjustments regarding bit size, pile wall size as well as founding level were adopted. The paper draws upon precautionary measures to mitigate adverse impacts.

1 INTRODUCTION

Queensway is one of the busiest trunk roads in Hong Kong. This new bridge connects Pacific Place with Harcourt Garden, integrating seamlessly into the existing pedestrian network and extending its reach to Harcourt Road and Tim Mei Avenue (Figure 1). Notably, it enhances accessibility to key locations like the Admiralty MTR interchange hub, government facilities and commercial buildings, significantly benefiting the neighborhood. The footbridge rests on 3 supports, namely Support “A”, “C” and “D”. Support “A” was at Pacific Place Mall. The ELS design for the pile caps for Support “C” and Support “D” was carried out. Mini-piles and shear H-piles were adopted as the foundation at these two supports.



Figure 1: Two Queensway Bridge in Hong Kong

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2 PROJECT BACKGROUND AND SITE CONSTRAINTS

2.1 Site description

The existing ground level of the Queensway is approximately +5.8 mPD, while Harcourt Garden is slightly above the Queensway. An existing ramp running down from Rodney Street to +2.8 mPD serves the Harcourt Garden basement carpark. As illustrated in Figure 2, the proposed footbridge is situated above a congested subsurface corridor with multiple existing underground assets in close proximity to the alignment. These include the Harcourt Garden basement carpark, a three-level underground facility founded on a combined raft and mini-pile system; an existing pedestrian subway linking Pacific Place One to Admiralty MTR Station; the MTR Finger Platform located at north of the proposed alignment; a further pedestrian subway connection to Three Pacific Place running along Queensway; and the Admiralty MTR Station Cavern underlying the site, with the shallowest crown level at approximately -21 mPD.

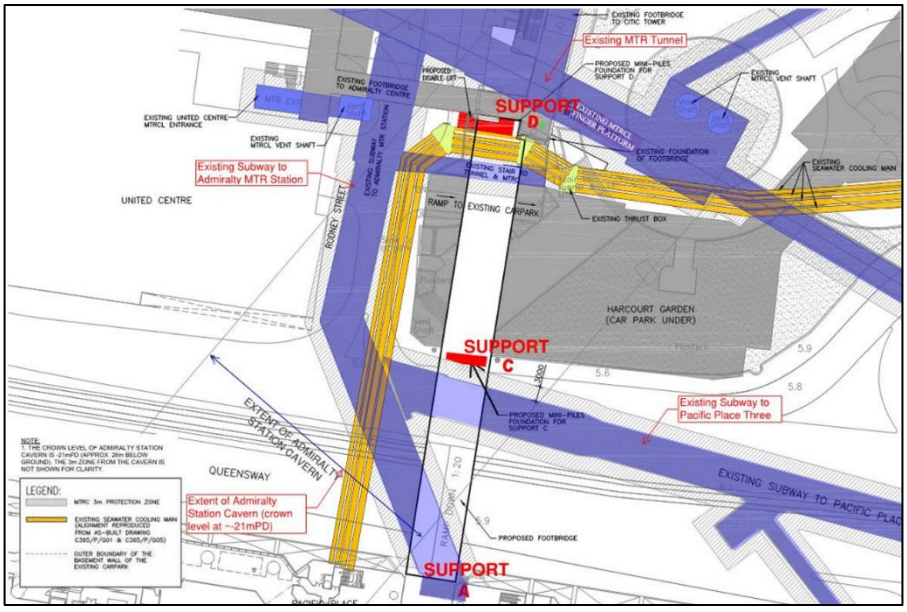


Figure 2: Existing underground structure

The proposed footbridge was constructed within a highly constrained corridor defined by existing transport and pedestrian infrastructure, which significantly increased design and construction complexity. Although the site was on reclaimed land with no registered man-made slopes or retaining walls along the alignment and the nearest recorded geotechnical feature was located more than 50 m away, the works interface directly with the existing Admiralty MTR station and entrances. The Admiralty MTR station occupied the footprint approximately bounded by the MTR Tsuen Wan and Island lines to the west of the existing CITIC Tower footbridge. In the vicinity of the connection between the proposed footbridge and the existing CITIC Tower footbridge, two MTR entrances were present. One entrance was along the Rodney Street, opposite to the Admiralty Centre and the other one located at the northeast corner of the United Centre. This imposed constraints and stringent requirements for safeguarding operational assets and passenger access. In addition, the new structure shall tie into multiple existing footbridges (CITIC Tower footbridge HF152 and the Admiralty Centre footbridges HF79 and HF164) that were founded on rock (grade III or better) via different systems (mini-piles and caissons, respectively). The locations of raking piles at HF152 were confirmed by prior trial pit investigations. These existing substructures and foundations restrict options for new support locations, constrain construction methodology, and necessitated careful coordination and detailed verification to avoid conflicts and ensure structural compatibility at the interfaces. The proposed footbridge was also near existing utilities. Drainage Services Department assets included a stormwater main and a foul sewer. Water Supplies Department infrastructure included a fresh water main and cooling seawater mains (2×Ø700 mm and 2×Ø1200 mm). A town gas pipe was present along the Queensway. Telecommunications services were extensive, with telephone cables, TV ducts, and plant/cables from PCCW-HKT Telephone Limited, Cable TV, Hutchison Global Crossing Limited, etc.

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2.2 Ground condition

The site geology was established through a review of (i) existing geological map showing regional geological information, (ii) existing ground investigation (GI) records, and (iii) as-built MTR project data for the Admiralty area. In accordance with Geological Map sheet 11 of the Hong Kong Geological Survey, the site is underlain by medium-grained granite. Relevant GI records were obtained from the Geotechnical Information Unit (GIU) of the Geotechnical Engineering Office (GEO). Because of the proximity of multiple structures near Supports "C" and "D" (Figure 2), only one borehole was drilled to characterize the underground stratum, supplemented by seven trial pits to confirm the locations of existing structures and utilities. The borehole was terminated after drilling at least 5 m into bedrock, comprising moderately decomposed rock of weathering grade III or better and a Total Core Recovery (TCR) no less than 85%. Laboratory testing was performed on rock samples to evaluate rock strength. The ground investigation, together with the existing borehole information, indicated that the site was covered by Fill material (0 to 4 m thickness) overlying completely decomposed granite of various weathering grade (4 to 7 m thickness).

3 ELS DESIGN

3.1 Design assumptions

For temporary excavation support during pile-cap construction, LSP-3B light-type sheet piles and FSPII-type sheet piles with steel grade S355 to BS EN 10025 (yield stress = 355 MPa, elastic modulus = 205 kN/mm²) were proposed as steel planking to support the excavation. Temporary shoring elements for ELS, including walers and struts, were Grade S355 to BS EN 10025. Ground investigation indicated that the site was typically overlain by approximately 2 m to 4 m of Fill, and the pile caps were proposed at about 3 m to 4 m below existing ground level. ELS installation and excavation works were anticipated to be undertaken predominantly within the Fill and partially within the Completely Decomposed Granite (CDG) layers. Upon obtaining no-objection from MTR, relevant ground investigation information from the nearby Admiralty MTR Station Cavern project was extracted and used herein for the interpretation of the soil design parameters. The interpreted soil parameters for Fill and Completely / Highly decomposed granite used for design and analysis are summarized in Table 1.

Table 1: Design soil parameters

Soil Stratum	Bulk Unit Weight, γ (kN/m^3)	Cohesion, c (kPa)	Friction Angle, φ (degrees)	Elastic Modulus, E (kPa)
Fill	18.5	0	35	6000
CDG/HDG	19	4	38	37500

No groundwater monitoring was carried out during the ground investigation. With reference to MTR's project data in Admiralty area, the groundwater level fluctuated between +3.0 mPD and -1.0 mPD in general with some spikes up to +4.0 mPD. A design groundwater level of +4.0 mPD was therefore assumed to conduct the structural design of the temporary sheet pile wall and shoring system. For settlement assessment, +3.0 mPD was considered representative, although +4.0 mPD was also applied in the settlement estimates to be conservative.

3.2 ELS support design

The ELS design was developed for construction of pile caps at Footbridge Supports "C" and "D", where existing ground levels are approximately +5.8 mPD and +6.0 mPD respectively. For support "C", the required excavation level for construction of pile caps supporting the footbridge is +2.125 mPD. A temporary light steel sheet pile wall with one level of strutting was proposed to provide stable support for the maximum 3.7 m deep excavation. Key site constraints were addressed in the design, notably the presence of numerous underground utilities identified from trial trench records, particularly along Queensway. Utility diversion within the pile cap footprint was to be completed prior to piling and excavation, except for an existing HKT concrete cable trough at Support "C". Relocating the HKT cables would require extensive diversion works, including breaking up of the concrete

trough and provision of a large and deep manhole to accommodate more than 40 cable joints. Due to these constraints, the HKT cable trough was retained with necessary protection works during the excavation and foundation construction of Support "C".

3.2.1 Construction sequence of support "C"

The section plot of Section 2 for Support "C" is shown in Figure 3. The construction at Support "C" was executed following a staged sequence developed to manage utility constraints and to maintain stability of the retained ground and the in situ HKT cable trough:

(1) Existing utilities within the excavation footprint were diverted as required, and the prescribed monitoring instrumentation was installed prior to commencement of earthworks.

(2) A temporary steel sheet pile wall was installed.

(3) Excavation was carried out in stages to approximately +5.1 mPD to expose the HKT cable trough.

(4) Foundation piles were then constructed within the retained excavation.

(5) A level of steel shoring/strutting was installed at approximately +5.6 mPD to control wall deformation during subsequent excavation.

(6) Excavation proceeded to about +4.2 mPD. Local trimming was undertaken to around +4.0 mPD near the support frame, and the HKT cable trough was temporarily supported/protected by a purpose-installed supporting frame.

(7) Excavation was continued to the final excavation level (FEL).

(8) The pile cap was constructed and the pier was erected to the extent permitted within the excavation. In other words, part of the pier at Support "C" was installed.

(9) Mass concrete backfill was placed up to the soffit level of the HKT cable trough, after which the temporary supporting frame was removed.

(10) Backfill was continued to approximately +5.3 mPD, followed by removal of the steel shoring/strutting.

(11) The remaining portion of the pier was constructed, and the excavation was subsequently backfilled to the existing ground level to complete reinstatement.

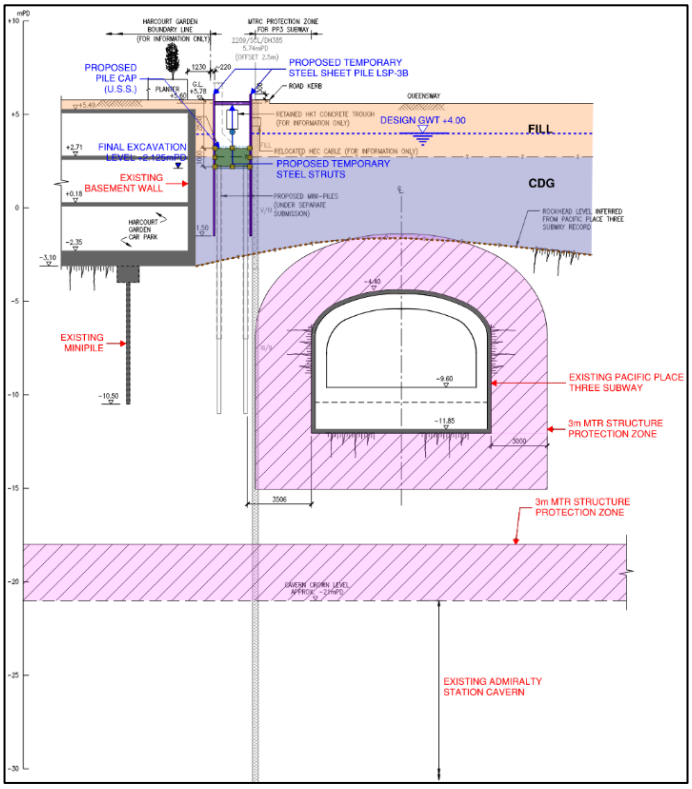


Figure 3: Section 2 of Support "C" (Section 2)

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3.2.2 ELS design process

Design surcharges applied on the retained side in the analysis included footpath loading (5 kPa), traffic loading (20 kPa), and construction loading (10 kPa), all treated as strip loads. Numerical analyses using PLAXIS 2D were performed to evaluate ground movements and wall and struts force. Regarding ground settlement estimation, the total ground settlement was due to sheet pile wall installation, groundwater drawdown and bulk excavation. The settlement due to sheet pile wall installation will be highly influenced by the construction method and workmanship on site. As the maximum driven length of sheet pile is 7.3 m in this project, it was assumed that the ground settlement due to sheet pile wall installation would be 3 mm, provided that good workmanship and installation control are maintained. For ground movement due to groundwater drawdown, with reference to MTR project data in the Admiralty area, the lowest measured groundwater level at Support "C" was below +0 mPD, which is below the proposed final excavation level of +2.125 mPD. Hence no ground settlement was anticipated as no dewatering work will be carried out below the lowest measured ground level. Finally, settlement induced by bulk excavation was significantly influenced by the depth of excavation, the stiffness of the wall, the arrangement and stiffness of the ELS structural systems, the magnitude of the surcharge, the stiffness of the soil both outside and inside the site, etc.

3.2.3 Discussion on the effect of the proposed ELS system to the existing structures

The existing structures near the proposed ELS works at Support "C" include: (i) Harcourt Garden Basement Carpark, (ii) Pacific Place Three Subway, (iii) Admiralty MTR Station Cavern, and (iv) existing underground utilities. The potential impacts of the proposed excavation on these structures were evaluated through numerical analyses using PLAXIS 2D. The calculation results for deformation and stress change are presented in Figures 4 and 5.

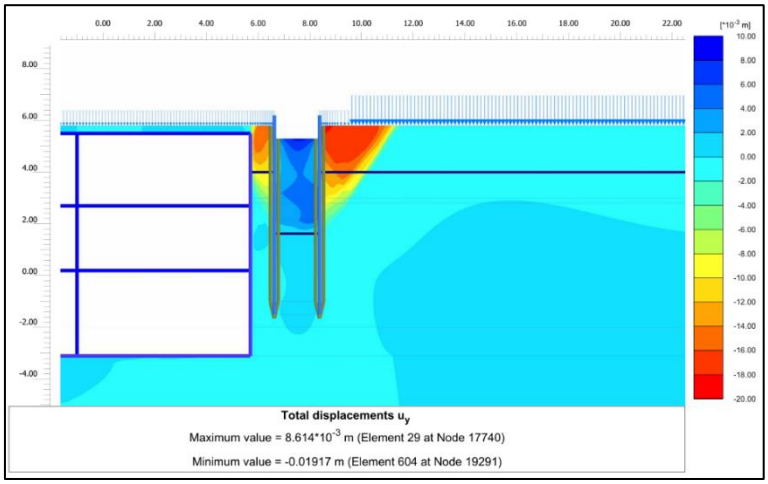


Figure 4: Contours of Total Displacement uy of Plaxis 2D Model for Support "C"(Section 2)

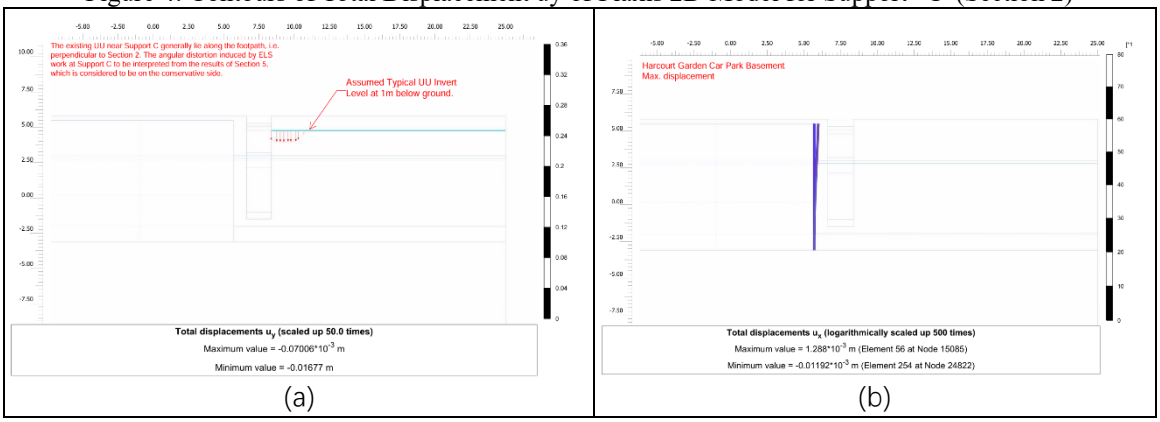


Figure 5a: Plaxis 2D Model Calculation Results of Support "C" (Section 2): (a) Predicted Total Displacement uy; (b) Predicted Total Displacement ux;

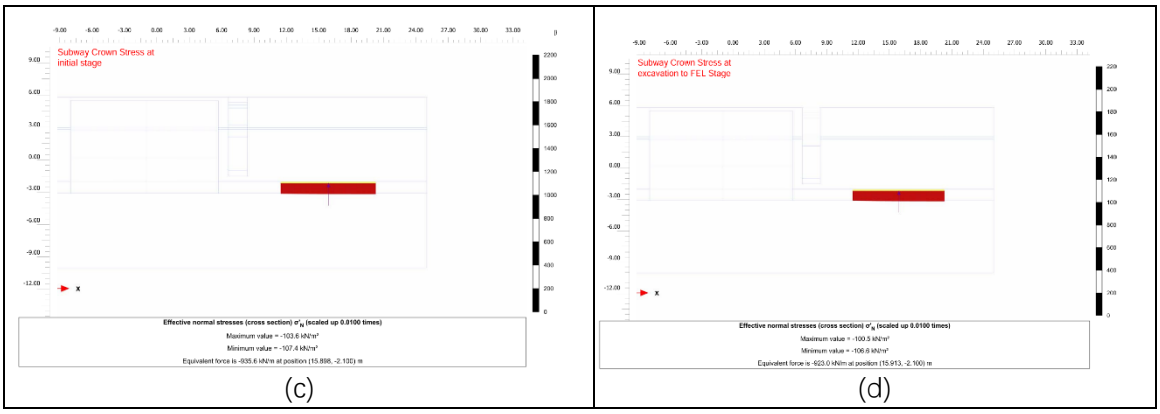


Figure 5b: Plaxis 2D Model Calculation Results of Support "C" (Section 2): (con't) (c) Predicted Subway Crown Stress at Initial Stage; (d) Predicted Subway Crown Stress at Excavation to FEL

(a) Harcourt Garden Basement Carpark: The existing Harcourt Garden Basement Carpark was located adjacent to the proposed excavation area at Support "C". Numerical results from Section 2 of the PLAXIS 2D analysis indicated that the predicted lateral displacement at the existing basement wall induced by the proposed excavation was negligible. Furthermore, the carpark was supported by deep foundations bearing at depth. Given the relatively shallow excavation depth of 3.7 m and the separation between the excavation and the deep founding levels, excavation-induced settlement at foundation level was considered unlikely. Accordingly, adverse structural effects on the basement carpark were not anticipated.

(b) Pacific Place Three Subway: The Pacific Place Three Subway was founded at levels between approximately –8.3 mPD and –17.45 mPD, with a structural height ranging from 5.85 m to 11.3 m. As-built drawings indicated that the subway structure is socketed into bedrock. The proposed ELS works will be undertaken entirely within soil and outside the 3 m protection zone (Figure 3) of the subway structure. Numerical modelling results demonstrated that the stress changes and deformation induced in the subway structure due to the proposed excavation were negligible. Therefore, the excavation works were not expected to impose any significant structural or serviceability impacts on the existing subway.

(c) Admiralty MTR Station Cavern: The Admiralty MTR Station Cavern beneath Support "C" is constructed within rock, with an estimated rock cover of approximately 19 m (rockhead level at –2.0 mPD and cavern crown level at –21 mPD). Considering the substantial rock cover and the fact that the proposed 3.7 m deep excavation and associated ELS system were confined to overlying soil strata, the influence of the excavation on the cavern was assessed to be insignificant. The numerical analyses confirmed that excavation-induced stress redistribution within the rock mass at cavern level was negligible.

(d) Existing Utilities: Trial trench investigations indicated the presence of multiple existing utilities within the footpath along Queensway, where Support "C" was proposed. Utilities that conflict directly with the excavation works will be diverted in advance. For utilities located adjacent to the trench, the predicted maximum ground settlement induced by the ELS works were less than the allowable serviceability settlement criterion of 25 mm, and the computed angular distortion was less than 1:300. These values were within commonly accepted serviceability limits for buried utilities. Accordingly, no adverse impact on adjacent utilities was anticipated. For the HKT cable trough to be retained across the trench during excavation, appropriate protective measures (e.g., installation of a temporary supporting frame) will be implemented to prevent undue disturbance and to maintain service continuity.

4 ASSESSMENT OF VIBRATION INDUCED FROM PILING WORKS

This section focuses on the assessment of vibration monitoring due to piling works using Peak Particle Velocity (PPV). The proposed footbridge over Queensway was designed to be founded on mini-piles and shear H-piles, which were to be installed using driven or pre-boring methods. According to Code of Practice for Foundation 2017, the ground-borne vibration due to the piling works can be estimated using the following empirical formula:

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$$V_{res} = k_p \left[\frac{\sqrt{W_e}}{r^{1.3}} \right] \quad (1)$$

where V_{res} = resultant PPV (mm/sec), k_p = ground bore vibration coefficient, W_e = nominal hammer energy (J), x = distance of recipient from pile measured along ground surface (m), y = depth of pile toe at the time of assessing V_{res} (m), r = slope distance of recipient from pile toe (m). More detailed discussion of the equation and the recommendation of fitting parameters were presented by Shang & Chen (2024).

4.1 Vibration resulting from H-pile installation by hammer method

The shear H-piles at Support "C" were driven using hammer / driven methods due to site constraints. As the driven length of the proposed shear H-piles is only up to 7 m, it was anticipated that the piles would pass through the fill and CDG layers. With reference to past project experience, k_p was assumed to be less than 0.5 for driving through the top soil layer with an SPT value generally lower than 20. Assuming a drop hammer weight W = 13 tons, a drop height H = 100 mm, and a hammer efficiency of 0.7, the estimated maximum vibration induced by the installation of the proposed shear H-piles is presented in Table 2.

Table 2: Vibration due to Hammer Method

Critical Pile	Critical MTR Structures	Minimum Clearance, r (m)	k_p	Hammer Energy, W_e (J)	Hammer Efficiency	PPV (mm/s)
CH4	Existing Subway to Pacific Place Three	4.547	0.5	12740	70%	6.6
CH1 to CH7	Admiralty MTR Station Cavern	19.700	0.5	12740	70%	1.0

4.2 Vibration resulting from pre-boring method

For shear H-piles at Support "D" and mini-piles, the concentric drilling method was adopted. Compared with the shear H-piles, the mini-piles were to be drilled to bedrock, which may be more critical in terms of vibration. Hence, the assessment of vibration resulting from pre-boring methods may need to be based on the kinetic energy of the DTH hammer used to form the rock socket of mini-piles. It should be noted that the vibration equation is empirical, and its application to pre-bored rock socketing is an engineering approximation. The kinematic energy of the DTH hammer piston and the applicability of equation (1) to pre-boring/DTH works were subject to preliminary assessment. The equivalence between piston kinematic energy and input energy required verification through site monitoring data. In other words, predictions for the pre-boring method were less reliable than for driven pilling, and attention shall be paid in engineering practice. In this study, for pile drilled to bedrock, the value of k_p was assumed to be 3.0 based on a first approximation according to Code of Practice for Foundation 2017. Assuming a hammer piston weight m = 68 kg, a piston movement frequency of 11.7 s^{-1}, and a stroke distance of 0.127 m, the estimated maximum vibration induced by the installation of the proposed mini-piles is presented in Table 3.

Table 3: Vibration due to Pre-boring Method

Critical Pile	Critical MTR Structures	Minimum Clearance, r (m)	k_p	Kinematic Energy of DTH Hammer, $We=(1/2)mv^2$ (J) *	PPV (mm/s)
C9	Existing Subway to Pacific Place Three	3.064	3	74.6	6.0
C1 to C12	Admiralty MTR Station Cavern	10.000	3	74.6	1.1

* The average velocity of the piston movement v = Frequency of the piston movement x Stroke distance

4.3 Precautionary measures for underground obstruction

Prior to the piling works, shallow obstruction shall be removed by the trial pit method. For underground obstruction beyond 3 m from the ground, it shall be removed by pre-boring method, such as coring or stitching coring using diamond drill bit with a slow advancing rate. For the H-piles at Support C, which were to be driven using the hammer method, precautionary measures were to be carried out to identify underground obstructions and prevent sudden increases in resistance due to their presence during piling works. At least one pre-drilled hole was to be made within the footprint of the pile cap prior to the piling works to check for any underground obstruction. Furthermore, the sheet pile wall was to be installed using a vibratory hammer or silent piler to support the excavation for pile cap construction. At Support "C", the sheet pile wall was to be installed before driving the H-piles, and the toe level of the sheet pile (-1.5 mPD) was slightly lower than that of the H-piles (-1.3 mPD). Hence the installation of the sheet pile would also help investigate the presence of underground obstruction for the following piling works of H-piles. The layout of sheet pile wall at Support "C" was presented in Figure 6:

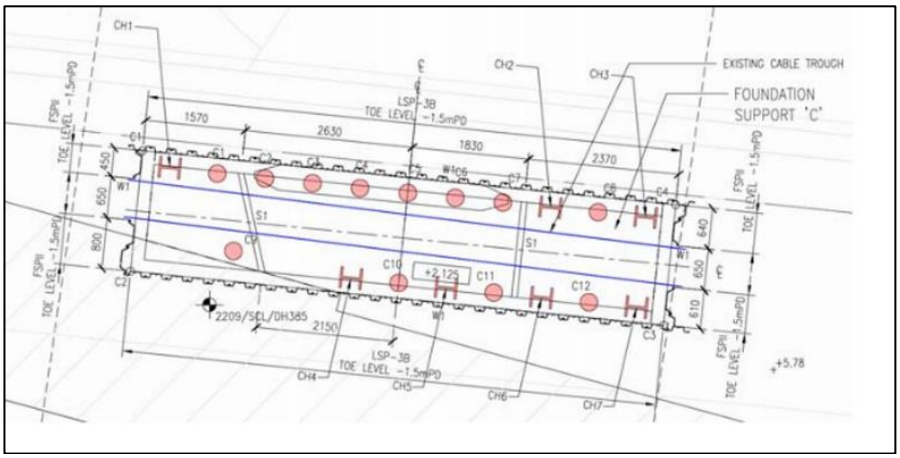


Figure 6: The layout of sheet pile wall at support "C"

In accordance with PNAP APP-24, the PPV induced in any railway structures resulting from driving or withdrawing of piles shall not exceed 15 mm/s. The estimated maximum vibration induced from the proposed piling works are less than the allowable limit, and it was therefore anticipated that there would be no adverse impact on adjacent railway structures.

4.4 Investigation report for vibration marker V8 during construction monitoring

Vibration monitoring equipment (continuous monitoring with daily review) was used to monitor the vibration induced during construction. The monitoring points are presented in Figure 7.

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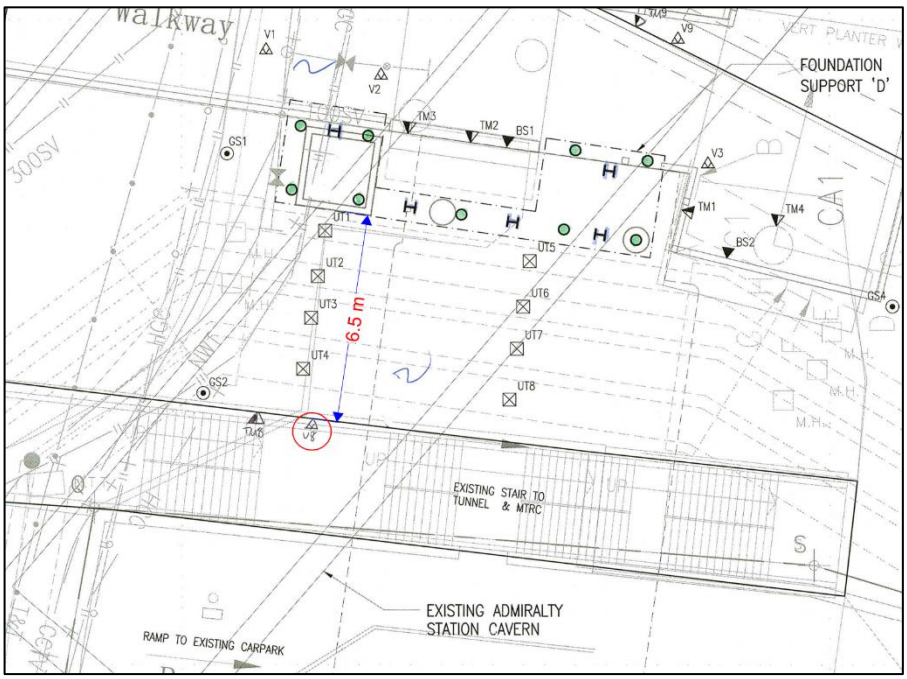


Figure 7: Approved monitoring plans

Up to 17 August 2023, no vibration reading had exceeded the Alert level except the vibration marker V8. The maximum PPV of 5.715 mm/s was measured at V8 on 26 July 2023. The measurement records are presented in Figure 8 and Table 4.

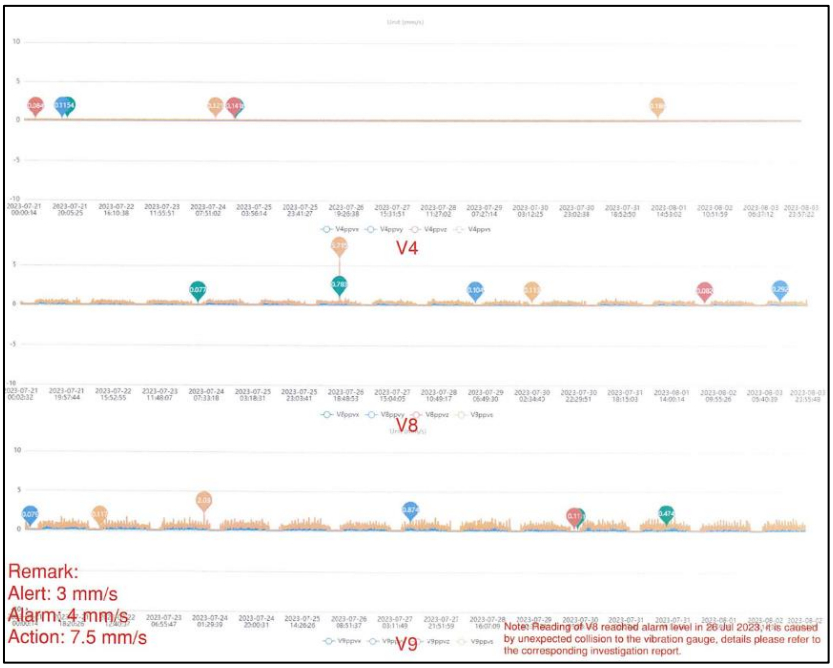


Figure 8: Excessive vibration recorded on 26 July 2023

Table 4: Summary of Vibration Monitoring (at MTR Structures) (from 21 July 2023-17 August 2023)

Check Point	Maximum Peak Velocity (mm/s)	Alert Level (mm/s)	Alarm Level (mm/s)	Action Level (mm/s)
V4	0.188	3	4	7.5
V8	5.715			
V9	2.401			

The contractor was requested to submit an investigation report. According to the report, the contractor suspected that the abnormal reading at V8 was caused by an unexpected collision with the vibration logger by passersby, as it was originally located next to a vending machine that was subsequently removed. For subsequent

monitoring works, warning signage was placed on the cover of vibration logger V8 to avoid similar incidents. The site record was presented in Figure 9.



Photo 3	Photo 4

Figure 9: Site condition photo record of vibration marker V8

Thus, it was concluded that the abnormal reading from vibration marker V8 was caused by unexpected collision of passersby for the following reasons: (1) No major site works that would induce high vibration occurred. The steady readings at V1 and V2 proved that the setup of the pile load test platform at Support "D" (18–21 July) did not generate significant vibration; (2) The vibration reading at V8 remained steady within the Alert/Alarm/Action level during the rest of the period. The preventive measures such as placing warning signage were done to prevent similar incidents occurring.

5 CONCLUSION

This paper presents a case study of ELS works for a footbridge constructed adjacent to Admiralty MTR Station and its associated underground structures in Hong Kong. Despite the highly constrained urban setting and the proximity of sensitive railway infrastructure, the combined use of numerical analysis, empirical vibration prediction, and site monitoring demonstrated that the works were carried out safely and within acceptable limits. The temporary sheet pile wall with single-level strutting provided adequate support for the excavation, while predicted and measured ground movements remained small and localized. Numerical analysis also indicated that the shallow excavation had negligible impact on the underlying MTR Cavern due to the presence of sufficient competent rock cover. For piling works, predicted and monitored vibration levels remained well below the relevant allowable limits, confirming the suitability of the adopted construction methods.

The deformation magnitudes and vibration levels documented in this study may serve as a practical reference for the design and construction of similar works in comparable ground conditions in Hong Kong. However, it is crucial to emphasize that the findings are specific to the geological context, excavation geometry, construction sequence, structural elements in the ELS system, and protective measures employed in the project. Direct extrapolation to other sites or conditions without independent judgement is not recommended.

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