Effects of Vibration due to Piling in Proximity of MTR Structure and Mitigation Measures

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ABSTRACT

Excavation and Lateral Support (ELS) techniques have been widely adopted in Hong Kong, where limited space and high population density necessitate innovative construction methods. Pile wall installation along the perimeter or within the site area for ELS works serves as common construction practice but invariably generates vibrations. Energy originates from the pile driver would cause the vibration of piles during the driving or installation process. Due to the interaction between pile and the surrounding soils, vibrations are transferred at the interface between pile and soil, and finally propagate through the ground, leading to vibration of ground and existing structures. The primary objective of this technical paper is to present vibration measurements during pile installation for temporary ELS retaining structure as well as permanent foundation and discuss the measures of mitigation for its influence on nearby buildings and MTR structures. It focuses on a specific case of ELS in Hong Kong, providing a detailed study based on actual site records, field experiments, and in-situ documentation of works. Additionally, the paper will draw upon other projects with vibration measurements to summarize the relationship between vibration and distance from the pile installation with consideration of various factors including ground conditions, pile type, ring bit size and piling methods, contributing to a deeper understanding of vibration-induced piling works. The study aims to offer valuable insights and precedents to engineering practitioners and researchers regarding vibration effects and corresponding mitigation measures.

Keywords: Vibration; Peak Particle Velocity; Pile Installation; Excavation and Lateral Support; Underground Structure; Mitigation Measures

1 INTRODUCTION

The proposed development is located at Central, Hong Kong, which comprises a 34-storey building with a single-level basement, reaching a height of approximately 153m. The site has approximate dimensions of 54m × 18m, with a total excavation depth of around 7.8m. The foundation works consists of 11 numbers of large dimeter bored piles. The site was previously occupied by Tak Shing House, which had an existing basement and was supported by pile foundation. The presence of the existing basement and foundation posed a significant challenge for the construction of the temporary retaining wall, permanent foundation, and basement of the new development. This site was also surrounded by One Chinachem Central building to the north, Des Voeux Road to the east, Wheelock House to the south, and Pedder Building to the west. The Pedder Building is a Grade I Historical Building. Additionally, the site falls within Scheduled Area No. 3 under the Building Ordinance. MTR Central Station Exits C and D, along with the island line tunnel, are in close proximity to the south and east of the site. The site location plan, including adjacent buildings and the MTR station, is displayed in Figure 1.

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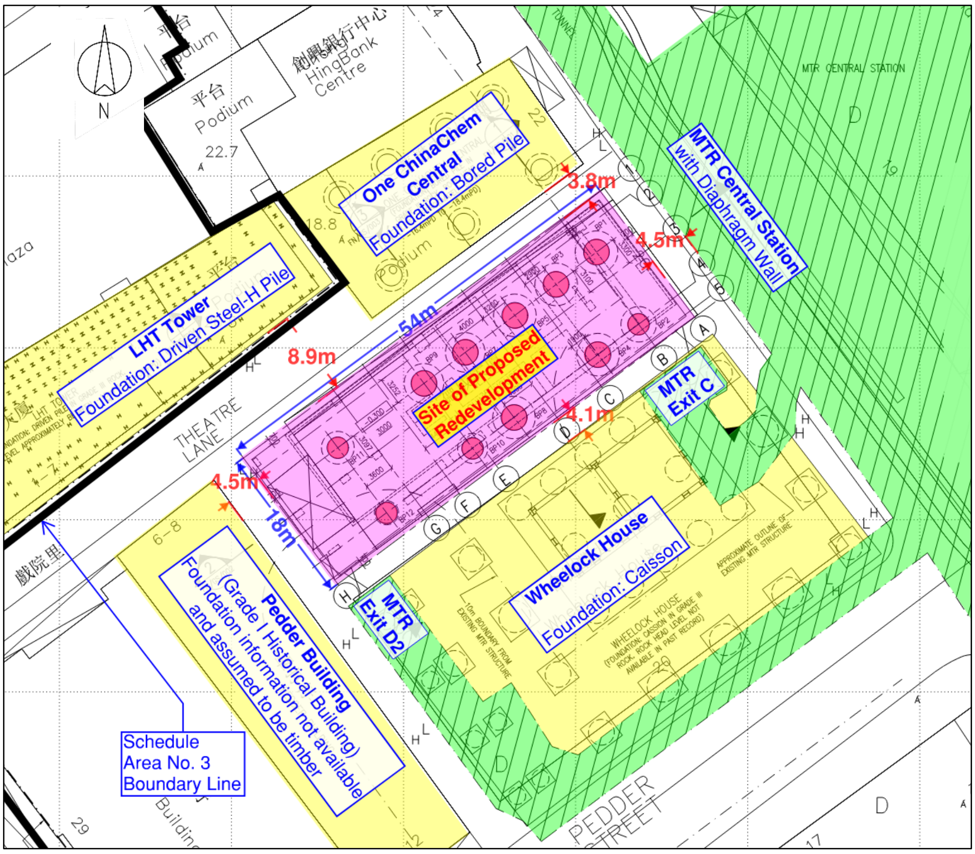

Figure 1: Site Location Plan

2 GROUND CONDITIONS

The ground investigation (GI) revealed that the geology of the site comprises fill, marine deposit, alluvium, completely/highly decomposed granite, and moderately/slightly decomposed granite. The geological condition is summarized in Table 1.

Table 1: Summary of Soil and Rock at Different Depths

Soil Material	Thickness (m)
Fill	4.0 to 6.1
Marine Deposit	2.0 to 8.0
Alluvium	2.0 to 8.0
Completely/Highly Decomposed Granite	3.1 to 9.0

The rockhead level of grade III or better rock generally varies from approximately -19.4mPD in the Northeast of the site to approximately -14.8mPD at the Southwest side. According to the groundwater monitoring record, the highest measured groundwater level is +2.940mPD.

3 DESIGN CONISDERATIONS

Laboratory tests were conducted on soil samples which were obtained from the GI works to determine the cohesion (c') and friction angle (φ'). The SPT N-values extracted from the vertical drillholes were utilized to estimate the elastic soil modulus. The effective soil stiffness, E (kPa), was calculated based on $E = f \times N \times 1000$, where the factor f ranges from 1.0 to 2.0 based on the soil type (Chan, 2003). In this project, f was conservatively assumed to be 1.0. A summary of the geological design parameters is presented in Table 2.

Table 2: Summary of Geotechnical Design Parameters

Soil	Unit Weight (kN/m^3)	Cohesion, c' (kPa)	Friction Angle, φ' (°)	SPT N Value Range	Adopted E (kPa)	Incremental E (kPa/m)
Fill	18	0	34	13-15	11,000	-
Marine Deposit	18	0	33	10-21	14,000	-
Alluvium (Sand)	19	0	36	10-53	18,000	-
CDG/HDG	19	5	36	20-200	27000 / 45000	7200 / 15400

The major constraints of the site were summarized as follows:

(1) The presence of weathered rock, corestones, and the existing basement wall near complicates the pile installation within the site;

(2) The requirements of the maximum pressure change (<20 kPa according to PNAP-APP24) and the maximum displacement for nearby historic building (with shallow foundation) and MTR structure pose additional challenges during pile installation works.

4 PROPOSED FOUNDATION AND ELS SCHEME

4.1 Foundation of Proposed Development

The proposed foundation system consisted of 11 numbers of large diameter bored piles varying from 2.5m to 3m in diameter, with founding level at Category 1(c) rock according to the Code of Practice for Foundations 2017.

4.2 Peripheral Wall and Construction Sequence of Basement

The proposed ELS works was to facilitate the construction of the pile cap, basement and superstructure. The general excavation level at Zone A, B and C was -3.5mPD, -2.5mPD and -5.2mPD respectively.

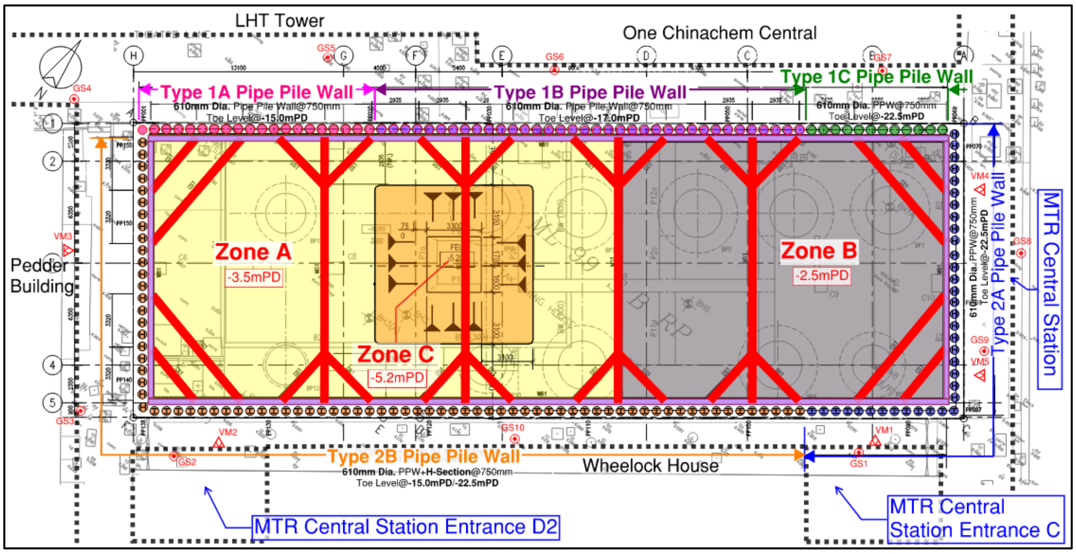

Figure 2: Proposed ELS Scheme – 610mm dia. Pipe Pile Wall and Three Layers of Struts

The excavation was supported by a CHS 610 x 30 pipe pile wall with a 305x305x283 UC at 750mm centres along the perimeter facing the MTR station, Wheelock House and Pedder Building whereas a CHS 610mm x 15.9mm pipe pile wall was proposed abutting the One Chinachem Central and LHT Tower. Grout curtain was proposed to provide the hydraulic cut-off. A total of three layers of steel shoring at levels +3.7mPD, +1.85mPD and -0.7mPD were proposed. A 200mm thick concrete propping slab was formed at the bottom of excavation for lateral support during removal of shoring and before cast of pile cap/basement.

5 VIBRATION ASSESSMENT IN DESIGN

Ground vibrations resulting from pile installation is a process of complex soil-structure interaction. Energy originates from the down-the-hole hammer during the drilling process would cause the vibration of piles during the installation process. Due to interaction between pile and surrounding soils, vibrations are transferred at the interface between pile and soil, and finally propagates through the ground, leading to vibration of the ground and the existing structures.

Wiss (1967) suggested that the magnitude of vibration was affected by the amount of energy, energy loss, soil properties and distance between source of vibration and the receiver. He suggested that the peak particle velocity (PPV) was a function of the square root of the hammer energy. Attewell and Farmer (1973) proposed the first empirical equation and suggested that the PPV was given according to the following equation. From filed measurements, they claimed that the results correlate quite well if k=1 and x=1. However, they suggested that k=1.5 should be used for practical conservative prediction of ground vibrations due to pile driving. The parameter k and x were discussed by many researchers to address the application of the empirical equation considering the effect of soil properties, pile types, source type, geometrical and material damping (Wiss, 1981; Attewell et al., 1992a; Wiss 1967; Hiller and Crabb, 1998).

$$PPV = k\left(\frac{\sqrt{W}}{r}\right)^{x} \tag{1}$$

where

PPV	Peak particle velocity
k	Empirically determined constant
W	Input energy
x	Empirically determined index

In this study, the PPV is estimated based on BS 5228-2:2009, TRL Report 429 (ASD SEB - PL 13, Section 5.2), and UK Transport Research Laboratory. Hiller et al. (2000) conducted specific ground vibration measurements for different construction equipment and process at a wide range of distances, and different types of ground conditions. The proposed empirical equation is widely used as the basis of construction vibration prediction. The following empirical equation as extracted from BS 5228-2 can be adopted to analyze the PPV due to pipe pile wall installation in Hong Kong according to Code of Practice for Foundation 2017:

$$PPV = k_p\left(\frac{\sqrt{W}}{r^{1.3}}\right) \tag{2}$$

$$W = W_h \times H \times \beta \tag{3}$$

$$\beta = K \times n \tag{4}$$

where

PPV	Peak particle velocity (mm/s)
k_p	Ground borne vibration coefficient
W	Hammer energy to pile, in joules (J)
r	Distance between source of vibration and the receiver (m)
W_h	Weight of hammer
H	Drop height (m)
β	Overall energy transfer
K	Hammer efficiency
n	Energy transfer from hammer head to pile

According to BS 5228-2: 2009, Hiller et al. (2000) proposed several empirical equations to predict the resultant peak particle velocities (PPV). Among several empirical predictors for ground-borne vibration arising from mechanized construction works, Equation (2) would be suitable for piling operation in this study. For other mechanized construction works such as vibratory compaction (steady state), vibratory compaction (start up and rundown), vibratory piling, dynamic compaction and tunnelling, other empirical equations would be more suitable. In this study, the pipe pile is installed by concentric ring bit drilling system, which reduces disturbance due to better air-flow and minimizes overbreak with specification of minimum advancement rates. The down-the-hole (DTH) hammer makes strokes within driver bit to have ring and casing penetrate through ground and it indicates that the mechanized construction works undergoes a process of percussive piling and the empirical equation is only affected by scaling factor k_p, nominal hammer energy W, distance between the vibration source and the receiver.

The scaling factor k_p is affected by the pile toe location, pile length, and ground conditions. Specifically, if piles are driven to refusal, the suggested value of k_p is 5. For piles not at refusal, the value of k_p is 3 if pile toe is driven through very stiff cohesive soils, dense granular soils, fill containing obstructions which are large relative to the pile cross-section. The value of k_p is 1 if pile toe is driven through soft cohesive soils, loose granular soils, loose fills, and organic soils (Hiller *et al*., 2000; BS 5228-2:2009). In this study, the scaling factor k_p is to be compared between values specified in table from Hiller *et al.*, 2000 and those calculated from back-analysis.

The exponent of 1.3 in the equation represents the power-law attenuation term for estimating peak particle velocity (PPV) as vibrations propagate from the source to the measurement point. This exponent is specific to the TRL (Transport Research Laboratory) reported by Hiller and Crabb (2000) and was derived based on empirical observations and measurements. The choice of the exponent value in the attenuation term depends on the nature of the source, the characteristics of the propagation medium (such as soil or rock), and the distance between the source and the measurement point. The exponent value is typically determined through field measurements and analysis of vibration data collected from different projects. The specific value of 1.3 in this equation indicates that the vibrations attenuate with distance in a manner where the amplitude decreases faster than a linear relationship (exponent of 1.0). In other words, as the distance (r) increases, the effect of the vibrations diminishes more rapidly due to factors such as energy dissipation and scattering in the propagation medium. It's important to note that the choice of the exponent can vary depending on the specific conditions and characteristics of the vibration sources and propagation paths. Different studies or reports may adopt different exponent values based on their specific observations and analysis.

While it is widely accepted that the vibration magnitude due to impact piling increases with increasing driving energy, there are insufficient database of different types of hammers to verify the relation in Hong Kong local practice. It is suggested to improve the database by analyzing the correlation between PPV and $\sqrt{W}/r^{1.3}$ considering the effect of ground conditions, levels of vibration, pile types, hammer types, etc.

In this study, two pipe piles (PP073 and PP090) were used to analyze vibration due to pile installation. Table 3 outlines the detailed calculation of the Peak Particle Velocity (PPV) for the proposed project for PP073 and PP090. The distance (r) between the vibration source and the receiver was approximately 4.6 meters. The scaling factor (k_p) was influenced by the depth of the pipe pile and was linked to the Standard Penetration Test (SPT) N value. Specifically, k_p was set at 1.0 for SPT N values below 80, at 1.5 for values above 80, and at 3.0 when the pipe pile reached the rock head level. Regarding the hammer efficiency coefficient (K), a value of 0.7 is assumed for the power machine being used (Poulos and Davis, 1980). The Hammer weight (W_h) was specified at 0.998 tons as per the manufacturer's catalogue and the maximum drop height was 0.2 meters according to piston stroke ($\leq$ 200mm) in specification of DTH. The energy transfer factor was taken to be 0.7 (McCabe and Flynn, 2017) as well as typical value from 0.7 to 1 in GEO Publication No.1/2006 based on general Hong Kong experience. With these parameters in place, peak particle velocities at different receiver levels were computed.

Table 3: Vibration Assessment for Pipe Pile Construction

Critical Case	Sensitive Receiver ntified building/ structure/ utility)	Plan distance between pipe pile and sensitive receiver (m)	Reciever Level (mPD)	r, slope distance between source and sensitive receiver (m)	Kp	Hammer type	Hammer weight, W_h (ton)	Max. drop height, H (m)	Hammer efficiency K	energy transfer factor η (efficiency of hammer blow)	Overall Energy Transfer (K x η)	W, hammer energy (KJ)	ppv (mms^{-1})
	MTR Central Station												
A1	Concentric ring bit drilling	4.550	+3.95	4.6	1.0	Piston	**0.998**	0.20	0.7	0.700	0.49	1.0	4.3
A2	Concentric ring bit drilling	4.550	+0.00	4.6	1.0	Piston	**0.998**	0.20	0.7	0.700	0.49	1.0	4.3
A3	Concentric ring bit drilling	4.550	-6.80	4.6	1.5	Piston	**0.998**	0.20	0.7	0.700	0.49	1.0	6.5
A4	Concentric ring bit drilling	4.550	-12.60	4.6	1.5	Piston	**0.998**	0.20	0.7	0.700	0.49	1.0	6.5
A5	Concentric ring bit drilling	4.550	-18.00	4.6	1.5	Piston	**0.998**	0.20	0.7	0.700	0.49	1.0	6.5
A6	Concentric ring bit drilling	4.550	-18.00	5.0	3.0	Piston	**0.998**	0.20	0.7	0.700	0.49	1.0	11.6

<Modified Alarm Level 10 mm/s

Refer to Piston Weight/Stroke in manufacturer's catalogue

<Modified Action Level 12.0 mm/s

🟨 **Pile installation in soil** 🟪 **Pile installation in corestone**

🟦 **Pile installation in rock**

6 IN-SITU MEASUREMENTS

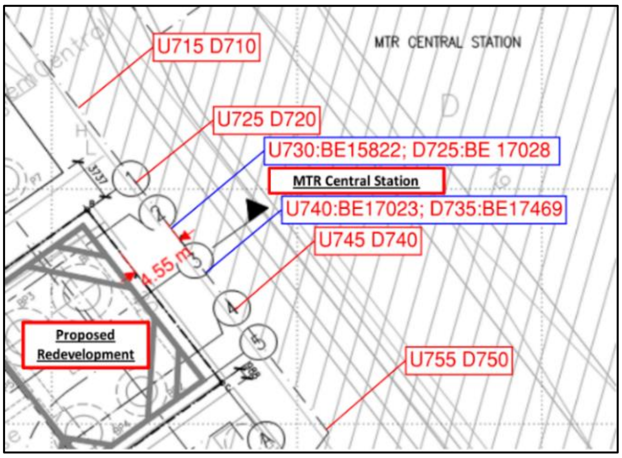

Figure 3: Vibration Monitoring along MTR Chainage

Plate 1: Vibration Sensor with Automatic Remote Data Transmission

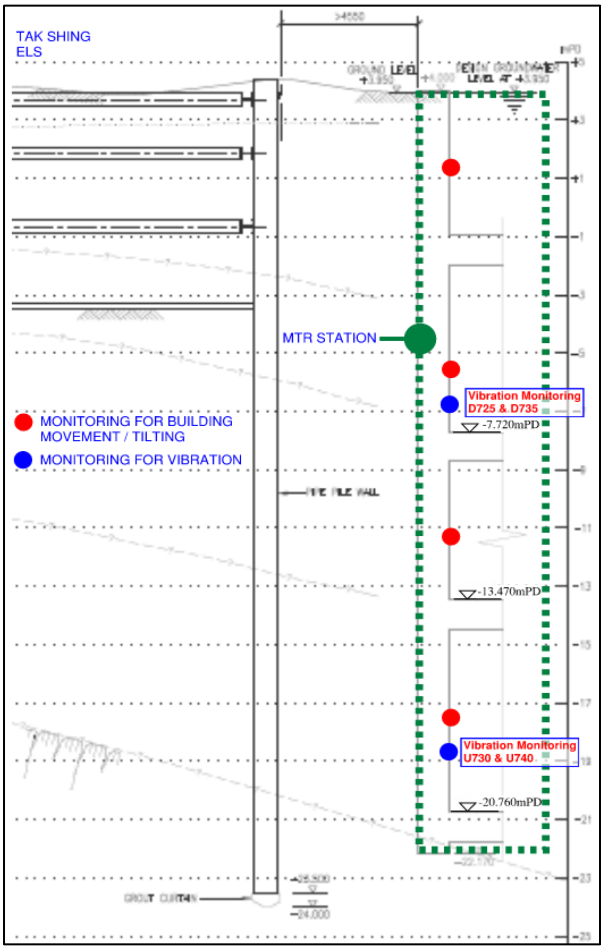

Figure 4: Location of Monitoring Checkpoints within MTR

Figure 3 illustrates the layout plan of the vibration monitoring points positioned along the MTR chainage. Figure 4 is a section plot showing the monitoring points' locations within the MTR station. For data analysis purposes, the vibration monitoring points D725, D730, D735, and D740 were used in this study. The elevations of points D725 and D735 were at -6.80mPD, while D730 and D740 were situated at -18.00 mPD. Additional information regarding the vibration sensors equipped with automatic remote data transmission can be found in Plate 1 for reference. It is worth noting that the pipe pile was embedded deeply into ground with toe level below Category 1(c) rockhead (~-22mPD), which is on purpose of preventing seepage resulted from high groundwater table (+2.94mPD) as well as controlling stress change during piling and excavation works within 20kPa (as required in PNAP APP-24).

6.1 Measured Vibration due to Foundation Installation

Due to the traditional methods and machinery adopted for installing large diameter bored pile (i.e. Hammer grabbing, Oscillator, RCD drillbit, Bell-out drillbit, etc), the vibration generated during foundation construction, according to vibration monitoring points on ground and within underground structure, was within tolerance and no larger than background vibration generated by pass-by vehicles and MTR train.

6.2 Measured Vibration due to Temporary Retaining Structure

Vibration during pile installation was closely monitored with different pile size, toe level, distance and ground conditions.

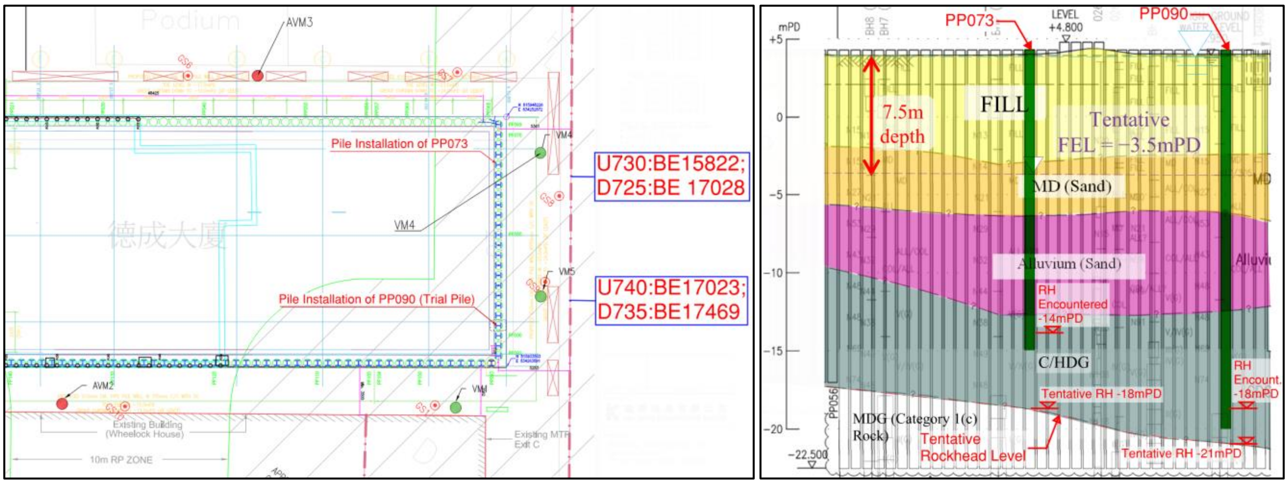

Figure 5: Location (Left) and Ground Conditions (Right) of PP073&090 for Vibration & Piling Level Relationship

Figure 5 shows the location of the pipe pile walls (PP073 and PP090) and the vibration monitoring points (VB-U730, VB-D725, VB-U740 and VB-D735). The receivers at VB-D725 and VB-D735 were situated at approximately -6.8mPD, while those at VB-U730 and VB-U740 were around -18.0mPD.

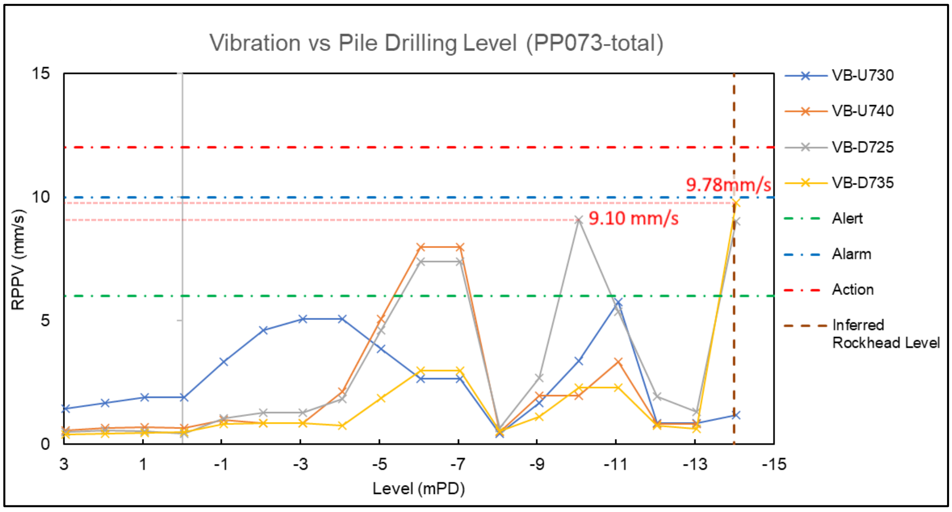

Figure 6: Vibration vs Piling Level for 610mm dia. PP073

Figure 6 illustrates the variation of PPV with the installation level during the piling works of PP073. The peak particle velocity for VB-D725 initially hit the alert level when the pipe pile was installed to around -6mPD, subsequently dropping to 0.5mm/s as the pipe pile was further driven to -8mPD. This could be attributed to the presence of hard materials (i.e. weathered rock, corestone, etc.) from -4.8mPD to -7mPD followed by a weak ground layer between -7.5mPD and -9.0mPD, with another potential weak layer between -12mPD and -13mPD following weathered rock/corestone near -10mPD. By referring to Figure 5 (Right) ground conditions and logging of nearby GI, it is found that soil stratum transits from MD composed of fine to coarse sand to Alluvium consisting coarse sand with subrounded fine gravel sized rock fragment at around -6mPD, which coincided abrupt increase of vibration measurement around same level. And similar case was obtained near -11mPD when geological layer changes from Alluvium to completely decomposed medium grained Granite. The peak particle velocities obtained from VB-D735 and VB-D725 as the pile approached rockhead near its founding level. It is worth noting that VB-U740 which is far away from PP073 also record drastic change of PPV during piling around -6mPD and the reason is likely due to vibration transmission within layer of corestone and between rock and MTR structure. Although the discrepancy is unavoidable, the plot is still valuable to provide a rough idea between the vibration and piling works and shall further our understanding regarding soil-pile interaction.

Figure 7 showcases the variation of peak particle velocity with the piling level during the piling activities of PP090, where a trial was carried out for evaluating the pile performance and vibration generated to nearby MTR structure. The peak particle velocity for VB-U740 triggered the alert level as the pipe pile was driven to -9.0mPD, decreasing as the pile reached -13mPD. Meanwhile, the peak particle velocity for VB-D735 (12.45mm/s) reached and exceeded the action level at -18mPD, which was close to the tentative Category 1(c) rockhead level. The pile installation was ceased immediately.

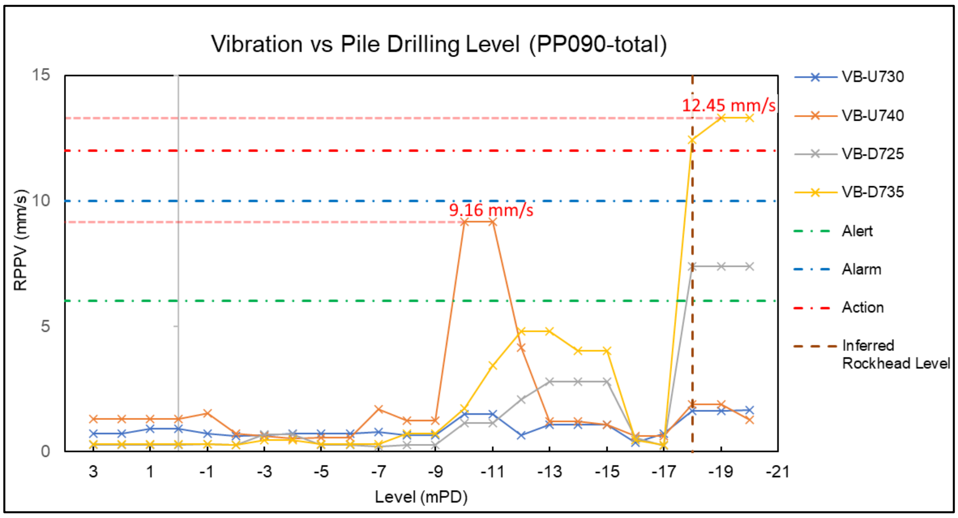

Figure 7: Vibration vs Piling Level for 610mm dia. PP090 (Trial Pile)

From Figure 6 and Figure 7, it can be observed that the Peak Particle Velocity (PPV) of receivers located far away from the pile remains more stable compared to those in close proximity to the vibration monitoring points. The increased fluctuation in PPV near the pile can be attributed to the presence of surface waves at the interface between the pile and the soils. The damping characteristics of the soils play a significant role in regulating energy transmission during piling activities, resulting in more uniform vibrations as the distance between the pile and monitoring receivers increases. Essentially, higher PPV levels near the pile are a result of surface waves interacting with the pile-soil interface. As the distance from the pile to the monitoring receivers increases, soil damping contributes to energy dissipation, leading to more stable vibration levels. This phenomenon stabilizes the magnitude and intensity of vibrations, as energy dissipates due to material damping and wave reflections across different ground layers.

Table 4: Correlation between Estimated Vibration and Actual Measurments

	ppv (mm/s)			**Average Diff.**	**k_p (original)**	**k_p (back-calculated)**
	Estimated	**Actual (PP073)**	**Actual (PP090)**			
vibration during pile installation in soil	4.3	4.62	3.45	-6%	1.0	1.0
vibration during pile installation in corestone	6.5	9.10	9.16	+41%	1.5	2.0
vibration during pile installation at rock	11.6	9.78	12.45	-4%	3.0	3.0

Assessment of effectiveness of Vibration Equation in Predicting in-situ Vibration

By comparing the estimated vibration with the in-situ measurements as shown in Table 4, the Equation (2) can be quite effective in predicting the vibration. However, effort needs to be further made in assuming appropriate values of W (with reference to manufacturer's catalogue, product specifications and project database) and k_p which would underestimate vibration level without appropriate verification by back analysis.

7 MITIGATION MEASURES

In response to the vibration monitoring point VB-D735 reaching the action level, mitigation measures were implemented to manage the vibration stemming from pile installation activities. From practical experience, adjustments were made to the dimensions of the pipe pile wall and the founding level to minimize vibration during the construction of the wall. Specifically, for PP057 to PP068, the pipe sizes were altered with CHS 610x30.0 (outer) + CHS 323x16.0 (inner) in replacement of the initial size CHS 610x15.9, with the toe levels of outer 610mm pile terminating at -15.0mPD and inner 323mm pile extending to -22.5mPD, where the bending moment was smaller than portion above, to mitigate vibration concerns. For PP069 to PP104, similar tube-in-tube pile wall was proposed to replace the CHS 610x30.0 with H-section, with the 610mm pile tentative founding levels at -15.0mPD and 323mm pile founded at -22.5mPD.

Figure 8 displays the vibration monitoring data of PP088 after taking mitigation measures, showcasing the effects of the modified pipe pile sizes and founding levels. Notably, VB-U740, VB-D725, and VB-U730 did not reaches the alert levels. However, VB-D735 did trigger an alert with a maximum peak particle velocity reaching up to approximately 7.6mm/s, possibly due to local hard materials. Comparing the PPV for VB-U740, VB-D735, and VB-D725 with those in Figure 7, the proposed mitigation measures are effective to control the ground vibration. It should be mentioned that the reduction of the pipe pile wall size could reduce the cross-section area of the interface between the soil and the pipe pile wall, contributing to reduction of the vibration.

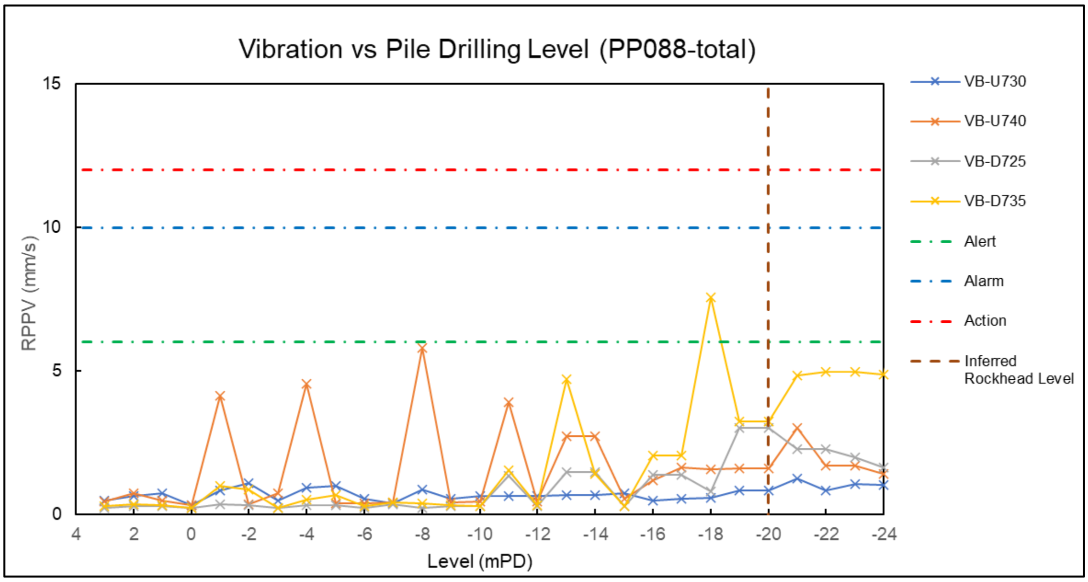

Figure 8: Vibration vs Piling Level for 323mm dia. PP088 (Trial Pile)

The pipe pile wall size and tentative founding level before and after taking mitigation measures are presented in Figure 9.

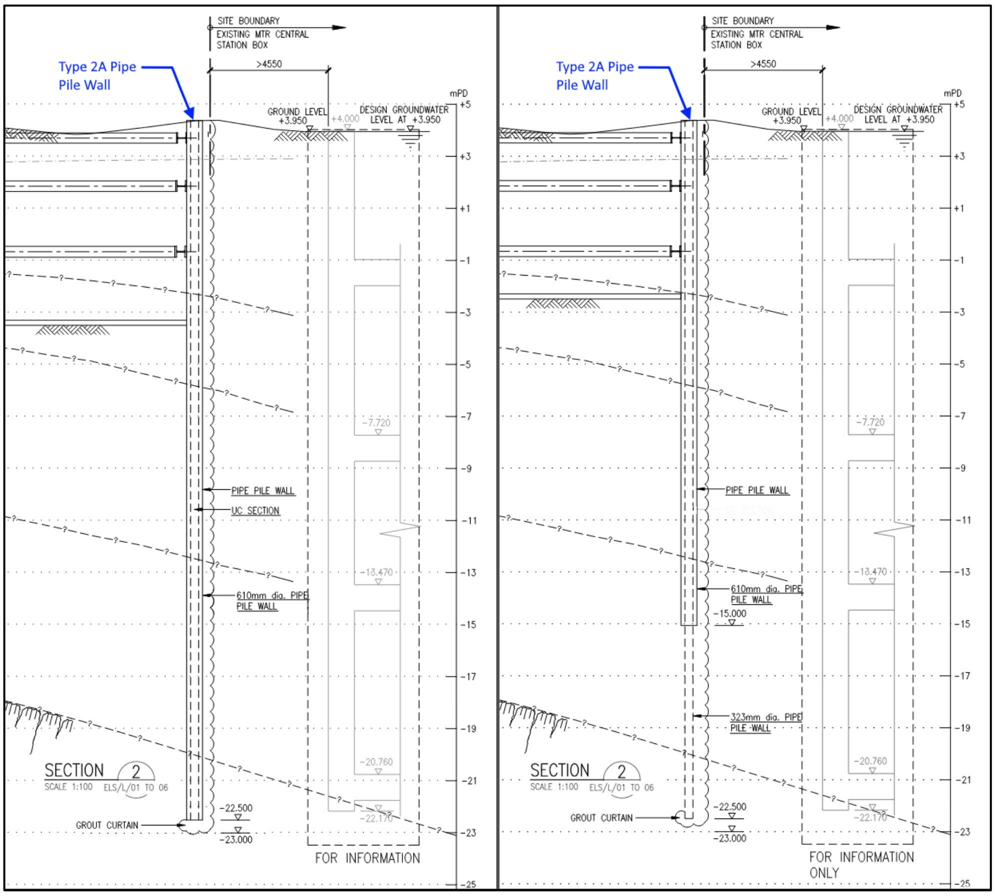

Figure 9: Change of Pile Wall from Casing with H-section (Left) to Tube-in-tube (Right)

8 REFERENCE TO OTHER PROJECTS WITH VIBRATION MEASUREMENTS

8.1 Case 1 – Project at Northwestern Hong Kong in adjacent to MTR

One project which can be referred to for the relationship between piling works and vibration is at northwestern area of Hong Kong. With sheetpiling works being carried out about 10m away from MTR rail track, the PPV of vibration monitoring is at 2.94 mm/s.

$$PPV = k_p\left(\frac{\sqrt{W}}{r^{1.3}}\right) \tag{2}$$

$$W = W_h \times H \times \beta \tag{3}$$

$$\beta = K \times n \tag{4}$$

where

PPV	Peak particle velocity (mm/s)
k_p	Ground borne vibration coefficient
W	Hammer energy to pile, in joules (J)
r	Distance between source of vibration and the receiver (m)

By assuming k_p at 1.0 for soil at ground (SPT N < 80) with hammer weight of 1 ton (W_h) to be dropped from 0.5m height (H),

Overall energy transfer:
$\beta = K \times n = 0.7 \times 0.7 = 0.49$

Hammer energy to pile, in joules (J):
$W = W_h \times H \times \beta = (1000 \times 9.81) \times 0.5 \times 0.49 = 2403$ J

Peak particle velocity (assumed $k_p = 1.0$):

$$PPV = k_p \left(\frac{\sqrt{W}}{r^{1.3}}\right) = 1.0 \times \left(\frac{\sqrt{2403}}{10^{1.3}}\right) = 2.46\ mm/s$$

The estimated PPV basing on Energy Equation is 16% smaller than the actual measurement (2.94 mm/s). And that is likely due to our adopting k_p lower than actual coefficient. Therefore, by re-adjusting k_p to 1.2 in comparing estimated results with field measurements,

Peak particle velocity (back-calculated $k_p = 1.2$):

$$PPV = k_p \left(\frac{\sqrt{W}}{r^{1.3}}\right) = 1.2 \times \left(\frac{\sqrt{2403}}{10^{1.3}}\right) = 2.95\ mm/s$$

The difference between actual measurement and estimated one basing on back analysis (0.2%) is considered satisfactory. Hence, the k_p value equal to 1.2 shall be adopted for vibration estimation under such ground conditions and construction arrangements of sheetpiling works.

8.2 Case 2 – Redevelopment at Mong Kok, Kowloon

The other project with vibration measurement is presented with predicted vibration (**Error! Reference source not found.**) and piling layout (Figure 10) below.

Table 5: Energy and Distance Estimation for Piling Construction

Vibration Receiver	Plan distance btw. pipe pile and vibration receiver (m)	r, slope distance between source and sensitive receiver (m)	k_p	Hammer weight, W_h (ton)	Max. drop height, H (m)	Overall Energy Transfer (K x η)	W, hammer energy (KJ)	ppv (mm/s)		Diff.
								Est.	Act.	
V1	14.90	17.4	1.5	0.998	0.2	0.49	1	1.1	1.17	+6%
V2	4.60	10.1	1.5	0.998	0.2	0.49	1	2.3	1.08	-53%

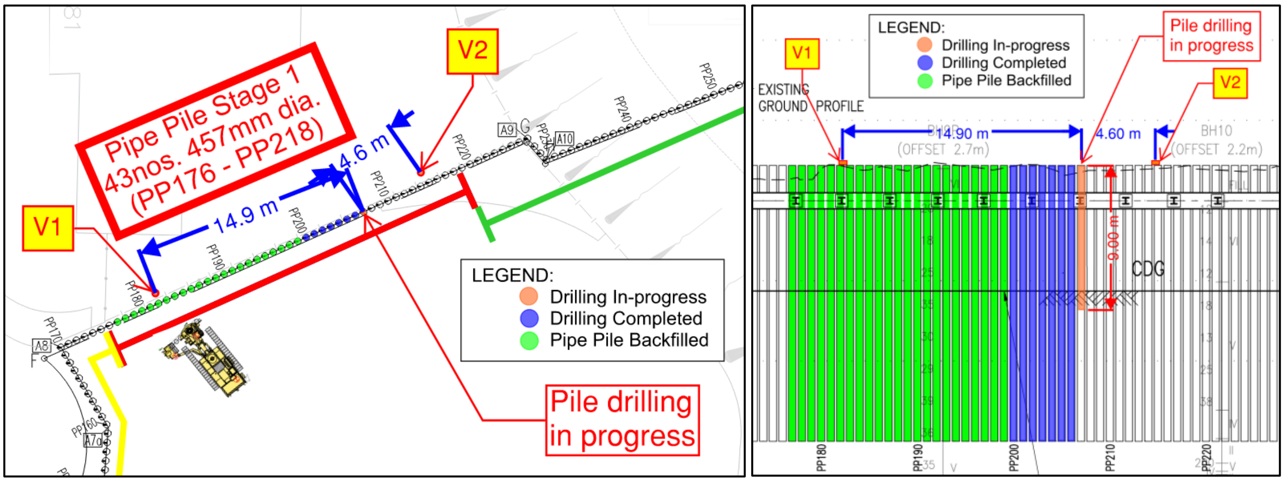

Figure 10: Location of V1 & V2 Relative to Piling Work in Progress

By comparing the vibration prediction with actual measurement in **Error! Reference source not found.**, the Equation (2) is effective in predicting V1 vibration. For V2, there are several possible reasons for discrepancy, including but not limited to instrument error, measurement gap and disturbance by other site works.

9 CONCLUSION

The vibration generated during pile drilling works is inevitable and target of engineering practice is to reduce its adverse effects. To have a sound estimation of vibration induced by piling works, reference has been made to empirical equation widely used which generates result reasonably predicting the actual vibration. The efforts made in assuming appropriate parameters, especially k_p varying from 1.0 to 1.5 for soil, 2.0 for corestone and 3.0 for rock obtained in our study, shall contribute to the professional's prediction in similar settings. To minimize the effects of vibration on the MTR structure, modified pile wall size with differential toe levels has been adopted basing on practical experience and sensitivity test. The outcomes are satisfactory with vibration within AAA limits. The mitigation results, along with reference to other projects with vibration measurements, provide engineering industry and academia further understanding of vibration-induced piling works and form basis for potential application of the measures.

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