GROUND MOVEMENTS DUE TO BORED PILES AND BASEMENT CONSTRUCTION

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ABSTRACT: Construction of bored piles and basement excavation both causes soil movements and can affect nearby structures and infrastructures. Measurements at various sites show that ground movements due to bored pile installation can be significant compared to basement excavation. Yet assessment of the effects of foundation and basement construction are focussed only with the latter. Ground settlement and vibration measurements during individual pile installation show low levels of vibrations and ground settlements that appear to be more pronounced for larger diameter and longer piles. Permissible movements are small and methods for reducing lateral movements of embedded walls have been developed. Assessment of the effects of basement excavation on nearby tunnels and piles has to be by use of three dimensional coupled effective stress analysis. Cases of three dimensional analyses are presented and results compared with measurements.

KEYWORDS: Ground movements, vibrations, bored pile installation, basement excavation, case histories, vibrations, three dimensional coupled effective stress finite element analysis

1. INTRODUCTION

It is necessary to limit ground movements during bored pile and basement construction to ensure against distress to adjacent structures. This is in fact the governing consideration for defining the criteria of permissible soil movements. Invariably assessment of movements involves only an assessment of movements during excavation most of the time using 2 dimensional (which at times is inadequate) but more recently 3 dimensional finite elements. Invariably no account is taken of the soil movements due to construction of bored piles and diaphragm walls. While the problems associated with driving piles affecting the surroundings are well documented, there is however little in the literature on soil movements due to bored pile installation.

Ground movements affect building foundations in different ways. Shallow foundations or short bored piles are more affected by ground settlement than longer piles. Slender long piles are more affected by lateral movements than larger diameter and stiffer piles. More likely older buildings are more susceptible to nearby foundation and basement works. It is unfortunate that in Malaysia there is no depository of information on foundations of structures. Older structures are founded on shallow foundations, smaller size and shorter bored piles, driven concrete piles and even piles that are formed by welding three or four steel rails to form triangular or rectangular piles.

The Malaysian Railway Regulations Act (1998) requires that the maximum movement of the tracks be kept to less than 15mm. This statutory limit is small as for example, 15mm corresponds to 0.1% of depth of excavation for a common basement excavation depth of 15m (three basement levels). Hence in many basement excavations close to railway structures e.g. MRT and LRT elevated structures and tunnels, it was found necessary to adopt measures such as buttresses, pre-stress struts and pre-stress floor slabs in top-down construction to minimize movements during basement excavation. This requirement of the Malaysian Railway Act which in essence demands 0.1% of depth of excavation for typical basements is more severe than the requirements of the Singapore Building Control (Amendment) Act 2012 and Regulations 2012: ERSS submission report.

It will be shown, by way of several case histories that soil movements during bored pile installation can be significant and the magnitude of ground settlement can be greater than over the basement excavation stage. Monitoring of ground movements and vibrations during installation of individual piles show that vibration levels are generally low. However accumulation of movements due to installation of large numbers of piles can result in significant ground movements affecting the surroundings.

Lastly the use of 3-dimensional coupled effective stress finite elements to simulate basement excavation and assess the effects of surrounding foundations are presented together with measurements.

2. CASE HISTORIES OF GROUND AND BUILDING SETTLEMENT DURING BORED PILE INSTALLATION AND BASEMENT CONSTRUCTION

Table 1 is a summary of the magnitude of bored piles and depths of basement excavation at 4 sites in Kuala Lumpur and Petaling Jaya. The methods of basement construction are similar viz. top-down construction. The larger volumes of piles are due not only to longer pile lengths but also to the greater area of the site and therefore the number of piles. The two sites with larger pile volumes and lengths of piles viz. Sites C and D are those where larger ground settlements over the period of piling were observed.

Table 1 Summary of bored piles at Sites A, B, C and D

SITES	**A**	**B**	**C**	**D**
Geological formation	**Granite**	**Highly weathered Kenny Hill over Limestone**		
Total volume of empty bore cu m	1231	3031	8741	17149
Total volume of bored pile cu m	2110	9236	14672	36657
Total volume pile borings cu m	3441	12267	23413	53806
Number of piles	174	85	97	313
Pile diameters, mm	600~ 1500	750~ 2800	1050~ 2000	750~ 3000
Average pile length (for different sizes), m	18.7~ 28.4	32.4~ 61.5	63.8~ 86.5	31.7~ 102.4
Material for backfilling empty bores	Loose soil from pile borings	Grade 10 concrete	Loose soil from pile boring	Grade 26 mortar
Excavation support system	Top-down	Top-down	Top-down	Top-down
Depth of excavation, m	13.1	16.8	22.9	23.6

2.1 SITE A

Figure 1a illustrates the soil profiles, basement levels and length of the diaphragm wall that is mostly shorter than the foundation piles that are socketed into Grades I and II granite bedrock. The dimensions of low stiffness buttresses as a means for reducing lateral movements are also shown therein.

Figure 1b illustrates the measured ground settlement beneath an elevated MRT line approximately 15m from the diaphragm wall with the measurements taken over the period of pile and diaphragm wall installation and subsequent basement excavation. The measured settlement was up to 28mm.

Variable but appreciable proportions of ground settlement occurred during foundation installation before start of basement excavation. The bored piles and diaphragm wall were constructed at the same time. Figure 1c shows that the proportions of ground settlement during pile installation range from 26.2% to 47.3% of the total settlement with an average of 33.8%.

The plot of the lateral movements from inclinometers at the depth where the lateral deflection is the highest is shown in Figures 1d, 1e and 1f; each graph comparing the lateral deflections at distance of 15m from the diaphragm wall and at the diaphragm wall itself. The lateral deflections during pile installation are significant. Figure 1g shows that the lateral deflection of the diaphragm wall during the period of pile installation amounted to about 35% of the total lateral deflection of the wall i.e. there was appreciable movement of the diaphragm wall over the period of pile installation and before excavation. At 15m away, the proportion of lateral deflection over the pile installation period vary significantly from about 16 to 62% of the total lateral deflection (see Figure 1h).

2.2 SITE B

The site is close to the interface between Kenny Hill and limestone.

Figure 2a illustrates the soil profile, secant pile lengths and basement levels. Most of the piles and part of the secant wall piles are socketed into limestone bedrock.

Figure 2b illustrates the ground settlement versus time. The total measured settlements are relatively small amounting to about 7mm at most. However the proportions of settlement over the period of bored pile installation range from 30 to 65% (see Figure 2c).

Figure 2d shows there was no draw-down of ground water over the period of bored pile installation.

Lateral deflections by inclinometers within the secant pile wall over the period of basement excavation range from 15 to 30mm.

2.3 SITE C

The site is close to the interface between Kenny Hill and limestone.

It appears from general local information that the adjacent building is on short, small diameter bored piles. Figure 3a shows the sub surface conditions as well as the contiguous bored pile wall length and depth to bedrock. The piles are mostly socketed into limestone bedrock and significantly longer than the contiguous bored pile wall.

Figures 3b and 3c illustrate the measured settlements of the ground and the building columns over the period of pile installation and basement excavation. The ground settlement over the pile installation period corresponds to at least 73% of the total measured settlement (See Figure 3d). Total ground settlement were up to 93mm and the building settlement close to 40mm. The ground settlement here is the highest of the 4 sites.

Figures 3e and 3f are plots of ground and building settlement with distance from the contiguous bored pile wall. Ground Settlements and with it building settlements extend to more than 70m from the line of the contiguous bored pile wall. Figure 3g shows there was no draw-down of ground water over the period of bored pile installation.

Unfortunately monitoring of the inclinometers did not commence until after completion of pile installation. Hence lateral movements during pile installation is not known. The magnitude of lateral movements over the period of basement excavation is of the order of 30 to 40mm.

2.4 SITE D

This site is also located close to an interface of Kenny Hill and limestone.

Figure 4a illustrates the sub surface conditions with length of the diaphragm wall and bored piles. The piles are mostly socketed into limestone bedrock and in places extended to lengths in excess of 100m. The total number of piles far exceeded the other three sites.

Next to this construction site is a building on shallow foundations.

Figure 4b illustrates the measured settlement at the adjacent building over the period of bored pile installation and basement construction. The measured settlements at a distance of about 25m from the site boundary is about 20 mm reducing to about 7mm at a distance of 70m (see Figure 4c). Figure 4d shows the proportion of the measured settlement during and after pile installation. 70% to 100% of the total settlement occurred during installation of the bored piles. Figure 4e shows absence of drawdown during the period of bored pile installation.

2.5 SUMMARY OF THE 4 SITES

Table 2 is as summary of the measured settlements at the 4 sites.

Table 2 summary of measured settlement

SITE	Maximum measured ground settlement (mm)	Proportion of settlement over period of pile installation	Proportion of settlement during basement excavation
A	28 (15m from diaphragm wall)	26 to 47%	53 to 74%
B	7 (close to secant pile wall)	30 to 65%	35 to 70%
C	93 (8m from cbp)	70%	30%
D	25 (15m from cbp)	70 to 100%	0 to 30%

3. MONITORING OF VIBRATIONS AND MOVEMENTS DURING BORED PILE INSTALLATION AT SITE E

3.1 GROUND SETTLEMENT DURING INSTALLATION

At a location referred to as SITE E in KL which is within a limestone formation with sandy overburden, bored piles were installed within 5m of an important infrastructure and monitoring of surface movements and vibrations carried out as the bored piles were installed. The piles were of diameters ranging from 1.8m to 2.5m and surface settlements were monitored up to 12.5m from the pile being installed. Table 3 is a summary of the conditions through which the piles were bored. Casings were up to a maximum of 15m length and rotated in place and the pile boring stabilized using bentonite. Pile lengths were up to 40m with significant length of

coring into rock. The settlements (see Figure 5) for all piles except 3 are less than 1.0mm. Ground settlements of between 3 and 8mm were recorded for 3 piles F, G and H. These are amongst the bigger (2.2 to 2.5m diameter) and longer piles (29.5 to 40.0m). It should however be noted that settlements for long piles such as Piles E (2.0m diameter and 39.5m length) and J (1.8m diameter and 39.5m length) are less than 1mm.

The magnitude of settlement during pile installation is similar to that calculated for diaphragm wall panel installation by Ng and Yan (1999). The effects of bored pile in over-consolidated clays are given by Clark, Powrie and Richards (2006).

It is conceivable that the sum total ground settlement from installation of a large number of bigger diameter and longer bored piles could be as large as that measured at Sites C and D.

Table 2 Pile diameter and pile lengths

Pile Number	Pile Dia. (m)	Length bored through soil (m)	Length bored through rock and grouted cavities (m)	Total pile length (m)
A	1.8	15.5	13.5	29.0
B	2.5	15.0	20.64	35.64
C	1.8	21.0	4.32	25.32
D	1.8	21.81	6.21	28.02
E	2.0	22.5	17.0	39.5
F	2.2	20.5	9.0	29.5
G	2.5	11.5	26.5	38.0
H	2.5	14.0	26.0	40.0
I	2.5	14.0	12.0	32.0
J	1.8	27.5	12.0	39.5
K	1.8	14.8	12.0	26.8
L	1.8	11.0	16.0	22.0

3.2 VIBRATIONS DURING INSTALLATION

BS 5228 Part 4 1984 contains data on levels of vibrations due to installation of smaller diameter piles. Figure 6 is a plot of the data from BS 5228. At a distance of 3 m from the pile, the peak particle velocities (ppv) range from 1 to 15 mm per sec. At a distance of 20m away, the ppv are all below 1 mm per sec. Vibration limits from DIN 4150 and BS 5228 are included therein.

Continuous vibration monitoring were carried out at a distance of 3.0m from the pile throughout the pile installation. Casings of up to 15m were installed and extracted by rotation.

Figures 7a to 7g are ppv pie charts for piles A, B, C, D, E, F and G of Table 2. With the exception of pile G, vibration monitoring showed that more than 90% of the measured ppv for each pile are less than 1.5 mm per sec. Pile G is installed next to a busy road and the higher vibrations may be due to traffic. The measured ppv values all at 3m distance are at the lower bound of Figure 6. The relatively small ppv measured indicate that vibrations is unlikely to be the cause of ground movements at distances from the piling site as discussed in Section 2.

4.0 AN "IN EXTREMIS" CASE OF COLLAPSE DURING CONCRETING AT SITE F

It is not uncommon for collapse to occur during pile borings. Of greater consequence to the surroundings is when there is a collapse during concreting. Such was the *"in extremis"* case of a pile formed over deep seated soft organic clays over limestone at an interface between limestone and Kenny Hill Formation at Site F. The pile diameter and drilled length of the pile was 1800 mm and 100m socketed 6m into limestone rock. In the midst of tremie concreting with 30m height of concrete above the pile toe, there was a sudden drop in the bentonite and concrete level; suspect that there was a collapse of the limestone below the toe of the pile. This led to an overall subsidence of the platform by about 500mm over an area of about 30m x 50m. The nearby diaphragm wall of 35m length translated laterally with the toe and top of the wall moving by about 25mm and 100mm respectively.

Some columns of surrounding high rise buildings within 10m distance of the diaphragm wall settled by 30mm in one case and 60mm in the other case. All these buildings were founded on drilled piers.

5.0 METHODS FOR REDUCING LATERAL MOVEMENTS IN A BASEMENT EXCAVATION

The Railway act stipulates a very tight criteria of maximum lateral movements of the track as 15mm. Given the absence of predictive tools on ground movements during installation of bored piles, it becomes necessary to tightly control the movements of the embedded wall during excavation to small magnitudes. Over the course of more than 20 years of working on basement excavations, the following methods have been developed:

(i) Pre-stressing with struts (see PLATE A and PLATE B);

(ii) Pre-stressing floor slabs being part of top-down construction (See PLATES C and D);

(iii) Low stiffness buttress buttresses (Concrete strength = 1 MPa) in the form of rectangular diaphragm wall panels perpendicular to the embedded wall line (PLATES E and F).

(iv) High stiffness buttress (Concrete strength = 10 MPa) in the form of diaphragm wall panels perpendicular to the embedded wall line (PLATES G and H).

6.0 METHOD FOR ESTIMATING EFFECTS OF BASEMENT EXCAVATION ON SURROUNDING STRUCTURE

Two dimensional finite element analysis is most commonly adopted for estimating lateral deflections, bending moments and shear forces of the embedded wall and ground movements behind the wall. The appropriate type of analysis for a basement excavation would be a coupled effective stress analysis given that the period over which the basement is constructed corresponds neither to undrained or drained conditions but one where excavation with reduction of stresses results in changes in pore water pressures and effective stresses and also changes to the seepage state.

However when assessing the effects of soil movements on an existing piled foundation, two dimensional analysis is inadequate for the following reasons:

(i) In two dimensional analysis, piles become walls of reduced modulus

(ii) Derivation of forces induced in the pile requires an assumption of Young's modulus of the "equivalent pile"

(iii) Movements of soil between the piles is not simulated.

A proper assessment of the effects of excavation on surrounding piled foundations and tunnels requires a coupled effective stress three dimensional analysis (Pang, Yong and Chow, 2005).

Where it is required to assess the effects of a basement excavation on a nearby tunnel, the three dimensional finite element analysis must include simulation of the tunnel construction.

7.0 ASSESSING THE EFFECTS OF BASEMENT EXCAVATION WITH LOW STIFFNESS BUTTRESS AT SITE A ON EXISTING MRT PILES

This involves excavation to a depth of about 13.1 m with 3 levels of basement (See Figure 1a). Figure 8a illustrates the finite element model with bored piles supporting the MRT piers together with a concrete drain on driven piles between the MRT line and the diaphragm wall. The MRT line was constructed long before the basement excavation and runs parallel to and about 15m from the diaphragm wall. Excavation was carried out in a top-down manner. MIDAS – GTS (version 2018) was adopted for the three dimensional analysis.

The subsoils take the form of up to 16 m of loosely deposited fill overlaying 12 m of hard layer before reaching bed rock level at about RL11m. Input parameters are given in Table 3. The initial ground water table was about 4 meters measuring from the ground surface.

Table 3 Parameters for analysis of SITE A

RL	Description	E_{50}	E_{oed}	E_{ur}	c'	ϕ'	k
m	**-**	**MPa**	**MPa**	**MPa**	**kPa**	**°**	**m/s**
39.5~ 26.4	Very loose silty sand	2	2	5	1	28	1e-7
26.5~ 23.4	Loose silty sand	10	10	25	5	28	1e-7
23.5~ 11.5	Hard sandy silt	50	50	125	5	35	1e-7
RL	**Description**	**E**	**k**	**-**	**-**	**-**	**-**
<11.5	Granite	7000	1e-9	-	-	-	-
-	Concrete	2.6e7	-	-	-	-	-
-	Buttress	3.6e6	-	-	-	-	-

The buttresses are 1.0m thick x 2.94m length at 8.5m centres. The concrete strength was 1 MPa. Inclinometers measured a maximum lateral deflection of 40mm without buttresses and 30mm with buttresses. The steps in finite element modelling follow the actual construction sequence as follows:

1. Initial ground stress
2. Excavate to RL37.8m for road construction
3. Apply 52kPa on the road
4. Construct RC piles, RC drain, plunged in column, bored piles and buttresses
5. Construct slab at RL39.45m
6. Excavate to RL34.55m
7. Construct slab at RL36.35m (B1)
8. Excavate to RL31.55m
9. Construct slab at RL33.35m (B2)
10. Excavate to RL26.35m
11. Construct slab at RL30.35m (B3)

Figure 8a illustrates the instrumentation layout plan and Figure 1a the cross section. Figure 8b shows the three dimensional model. Figure 8c and 8d shows the lateral deflection profiles for the areas with buttresses where the measured maximum lateral deflection is 30mm. Figure 8e shows the lateral deflection profiles the areas without buttresses where the measured maximum deflection is 40mm. Figure 8f compares the lateral deflection profile at the MRT line about 15m from the Diaphragm wall. The estimated deflection due to excavation is about 7mm compared to the total measured (over the piling and basement excavation period) of between 7 and 15mm.

The computed lateral deflection profiles appear to compare well with the measured profiles. However at the depths of maximum lateral deflection, approximately 10mm had already taken place before start of excavation (see Figures 1d and 1e). Therefore the finite element analysis slightly overestimates the deflection during excavation.

8.0 ASSESSING THE EFFECTS OF BASEMENT EXCAVATION WITH DIAPHRAGM WALL AND HIGH STIFFNESS BUTTRESS AT SITE G

Bored piles were not adopted at this site and instead the foundations take the form of jack-in-place piles. This is a two basement level excavation with maximum excavation depth of 10.2 m. An MRT station and MRT piers were completed before the basement excavation. Both the MRT station and piers are founded on bore piles. The nearest station piles (600mm diameter) are at a distance of 13m from the diaphragm wall whereas the pier is 40m away from the diaphragm wall. The thickness of the diaphragm wall is 600mm. Figure 9a illustrates the site plan showing the diaphragm wall, buttresses, U shape strut slabs and inclinometers. Figure 9b illustrates the cross section. The buttresses are 600mm thick x 6.0m length and spaced at 8.4m centres. The concrete strength is 15 MPa. Excavation was carried out in a top-down manner. Ground water table was about 6 meters below ground surface. Soil input parameters are given in Table 4.

Table 4 Parameters for analysis of SITE G

RL	Description	E_{50}	E_{oed}	E_{ur}	c'	ϕ'	k
m	**-**	**MPa**	**MPa**	**MPa**	**kPa**	**°**	**m/s**
29~4	Loose silty sand	8	8	20	5	28	1e-7
RL	**Description**	**E**	**k**	**-**	**-**	**-**	**-**
<4	Granite	8000	1e-9	-	-	-	-
-	Concrete	2.6e7	-	-	-	-	-
-	G15 Buttress	1.8e7	-	-	-	-	-

The steps in the finite element analysis follow the actual construction sequence as follows:

1. Initialization of ground stress
2. Construct bored piles, pile cap, diaphragm wall, buttress, reset displacement to zero
3. Excavate to RL24.4m
4. Construct slab at RL24.4m
5. Excavate to RL18.8m
6. Construct slab at RL20.8m and construct RC walls
7. Construct slab at RL28.6m
8. Backfill soil behind retaining wall
9. Consolidation for 30 years

Figure 9c illustrates the finite element model. Figures 9d, 9e, 9f, 9g, 9h and 9i illustrate the computed and measured lateral deflection profiles for the diaphragm wall parallel and perpendicular to the MRT line.

The estimated maximum deflections at Inclinometer 1 and Inclinometer 2 perpendicular to the MRT line is about 10mm more than measured. However measurements at Inclinometer 3, 4 and 5 within the diaphragm wall in front of the MRT station were very close to that measured with a maximum lateral deflection of close to 15mm. Readings at Inclinometer 6 located behind the diaphragm wall started before installation of the diaphragm wall; approximately 4 mm movements were recorded during installation of the diaphragm wall panels. The total measured lateral movements including installation of diaphragm wall and basement excavation was about 4 mm, slightly lower than computed.

9.0 ASSESSING THE EFFECTS OF BASEMENT EXCAVATION ON EXISTING TUNNEL. SITE H.

There is an existing intervention shaft for the MRT Twin tunnels next to Site H. The depth of excavation for the intervention shaft is close to 33m and constructed by strutting against 1180mm diameter secant pile walls. After completion of the intervention shaft and with the MRT line operating a development involving a basement that extends to depth of 10m was to be constructed. One part of the development involves sharing a common secant pile wall with the intervention shaft. Figure 10a illustrates the plan showing the line of the twin tunnel (upper at 17.3m depth and lower at 30.7m depth) each of 6.6m diameter, the intervention shaft and the proposed development.

The other embedded walls for the development involved strutting against 800mm thick diaphragm wall and 880mm secant pile walls along the line and offset about 6m from the MRT tunnels.

Figure 10b shows the typical cross section of the intervention shaft and tunnels. The overburden soil is mainly sandy silt with SPT N less than 6; rock levels range from 9 m to 42.7 m below ground level. The Karst formation was modelled in 3D finite element by 9 zones with different rock levels. The water table is 3.0 m below ground level. Soil input parameters are given in Table 5.

Table 5

RL	Description	E_{50}	E_{oed}	E_{ur}	c'	ϕ'	k
m	**-**	**MPa**	**MPa**	**MPa**	**kPa**	**°**	**m/s**
37~23	Loose silty sand	2	2	5	3	28	1e-7
23~ varies	Loose silty sand	6	6	15	5	28	1e-7
RL	**Description**	**E**	**k**	**-**	**-**	**-**	**-**
-	Limestone	27000	1e-9	-	-	-	-

The sequence in the analysis follows the construction phases listed below:

(a) PHASE 1 – Tunnel construction
  (i) Step 1. Initialize ground stress and apply 20 kPa surcharge.
  (ii) Step 2 to 124. Tunnel construction was simulated using the grout pressure method

(b) PHASE 2 – Intervention shaft construction (Steps 125 – 147)
  i) Construct IVS2 secant bored pile wall (SBP)
  ii) Excavate to RL35m
  iii) Install strut at RL35m
  iv) Excavate to RL31m
  v) Install strut at RL31m
  vi) Excavate to RL27m
  vii) Install strut at RL27m
  viii) Excavate to RL27m
  ix) Install strut at RL27m
  x) Excavate to RL23m
  xi) Install strut at RL23m
  xii) Excavate to RL19m
  xiii) Install strut at RL19m
  xiv) Excavate to RL15m
  xv) Install strut at RL15m
  xvi) Excavate to RL12m
  xvii) Install strut at RL12m
  xvii) Excavate to RL9m
  xix) Install strut at RL9m
  xx) Excavate to RL6m
  xxi) Install strut at RL6m
  xxii) Excavate to RL1.3m
  xxiii) Construct base slab, construct RC wall, slab and remove temporary struts and walers.

(c) PHASE 3 Basement excavation Stage 148 - 157)

  (i) Install diaphragm wall and SBP wall and bored piles
  (ii) Excavate to RL34.6m
  (iii)Install strut at RL34.6m
  (iv) Excavate to RL31.3m
  (v) Install strut at RL31.3m
  (vi) Excavate to RL28m
  (vii)Install strut at RL28m
  (viii)Excavate to RL25.95m. Excavate to RL21.875m and to RL23.575m
  (ix)Remove temporary struts and construct floor slabs
  (x) Apply loads on bored piles

The grout pressure method described by Moller (2006) was used to simulate the TBM tunnelling. Construction speed of 10 m per day was assumed in the analysis. Figure 10c illustrates the 3D finite element model. Inclinometer readings taken over the period of the intervention shaft construction enabled comparison with calculated movements at the end of PHASE 2 (see Figure 10d). The computed compared reasonably well with the measured.

Figure 10e illustrates the deformed shape of the embedded walls and tunnels. The analysis shows that the tunnel would deform about 2 mm towards the basement excavation. Figure 10f to 10h show that the axial forces of tunnel linings would increase by 150 kN/m to 200 kN/m. The increase in bending moment of the tunnel linings is negligible. Figure 10i and Figure 10j show the deflection in the shared wall to be negligible but the axial load in the shared wall has increased by 750 kN/m.

10. CONCLUSIONS

Ground movements are due to bored pile installation as well as basement excavation.

It is necessary to minimize ground movements to mitigate risks to surrounding structures and infrastructures. The risks are greatest for older buildings that are often founded on shallow foundations, short or slender piles.

Monitoring at 4 different sites show significant variation in ground movements at the surroundings during bored pile installation, from less than 10mm to 100mm. This limited study of only 4 sites appear to show that the problem is greater when the volume of pile borings are larger. Measurements also showed that the settlement during bored pile installation can be larger than during basement excavation. The induced ground settlement can extend to a distance of 70m.

Monitoring of ground settlement and vibrations during installation of individual bored piles showed vibration levels to be very low (casings are rotated in and out) and unlikely causative of the settlement problem. The limited data showed that ground settlement were larger for the bigger piles and longer piles. It is conceivable that superimposition of the settlement of many larger and longer piles could have resulted in significant ground settlements as observed.

Type of backfill material for empty bores may also have some effect on ground movements.

The author is not aware of any established procedure for estimating the ground settlement due to installation of bored piles. Yet it is important to be able to do so as legislated permissible movements are severely small.

Coupled effective stress three dimensional finite element analysis are able to provide a reasonable assessment of the effects of basement excavation and tunnelling on existing piles and vice versa.

The legislated magnitude of movements of rail tracks are severe amounting to 0.1% of not uncommon depth of excavation of 15m.

In view that it is presently not possible to estimate the soil movements due to bored pile installation with any degree of certainty, it will be necessary to adopt extra means to reduce movements during basement excavation such as pre-stressed struts, pre-stressed floor slabs, and temporary buttresses.

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