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Deflection Behaviour of GFRP Bar Reinforced Concrete Passive Bored Pile in Deep Excavation Construction

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ABSTRACT: This paper describes the investigation of a glass fibre reinforced polymer bar (GFRP bar) as a replacement for a traditional steel bar reinforcement in bored concrete piles with specific application to deep excavation construction. These two concrete passive piles were cast and experiments were conducted with reference to soil excavation. Normally, the point load which is applied to the pile head is provided by static lateral load test equipment for determining the pile behaviour; however, these two piles suffered from changed earth pressure during excavation. The amount and location of horizontal movement was monitored along the pile length by an inclinometer system which contained a PVC tube and a readout probe. The deflection behaviours of GFRP piles during the installation of one concrete and two steel supports were provided. It is concluded that, based on the difference between the total accumulated deflection of each pile, the GFRP bar reinforced concrete piles can resist the lateral loading and can provide an alternative to traditionally reinforced concrete piles used in shield construction.

KEYWORDS: Deflection behavior, GFRP bar, Passive piles, Deep excavation, Soil movement.

1. INTRODUCTION

Although utilization of piles is widespread, there are issues when these piles are located in harsh environments, especially in marine or coastal conditions. Piles made with traditional materials can be destroyed due to the corrosion of steel, the deterioration of timber, and the degradation of concrete. The deterioration of the timber, concrete, and steel piling systems costs the United States nearly 2 billion dollars per year for repair and replacement (Hassan & Iskander, 1998). The development of concrete piles has continued for more than 200 years, but engineers are now facing pile-related problems because, even though piles are made of concrete and steel, which has good rigidity and high strength, damage can still occur.

Fibre-reinforced Polymer (FRP) is well-known for its high ratio of strength to weight, high ratio of longitudinal/transversal Young's and high ratio of longitudinal/transversal shear modulus (Pecce, 2001). This material can be either Carbon-FRP (CFRP), Aramid-FRP (AFRP) Glass-FRP (GFRP), or Basalt-FRP (BFRP), depending on the fibre used. The composition of the FRP product is, therefore, flexible depending on the material properties and the volume ratio of fibres to resin, and the selection of both types and orientation of the fibre. Because of this flexibility, FRP composite material products have been widely applied for reinforcement and rehabilitation. FRP material can be manufactured into FRP laminar (slice) or FRP bars. The FRP bars can not only be used to replace steel bars inside concrete structure, but also can be used as anchors for slope reinforcement and support in deep excavation. This technology of using GFRP reinforcement as an anchor was adopted in the 2008 Chinese Changji Expressway construction built for red sandstone slope reinforcement. The results demonstrated the slope was overall stabilized (Luo, 2014). However, when rehabilitation is required, on either upper structures such as beams, or on columns, and sometimes on walls, the FRP laminar is utilized. The application of the FRP slice using resin onto the surface of the cracked beam or wall on the tensile side recovers the original strength of the components because the FRP material provides good tensile resistance.

The history of applying FRP piles is approximately three decades old. Dating back to April 1987, the first prototype recycled pile was driven at The Port of Los Angeles to replace the creosote-treated timber piles which successfully avoided any threat to marine borers (Juran, 2006). As early as 1998, the Empire State Development Corporation (ESDC) undertook a waterfront rehabilitation project known as the Hudson River Park (Robinson & Iskander, 2008). This project involved replacing up to 100,000 bearing piles for lightweight structures. The concrete-filled FRP composite piles were then employed by the Virginia Department of Transportation (VDOT) in 2000 for the new Route 40 Bridge over the Nottoway River in Sussex County, Virginia (Pando et al., 2004).

Extensive research studies have been conducted considering different types of FRP piles. Based on different loading conditions, various test equipment has been used to determine the bearing capacity of piles. For specific piles suffering from vertical loading, full-scale experiments were conducted to research FRP composite piles (Luo, 2014). This research interpreted the Load-Settlement curve based on Davisson offset limit method, DeBeer method and Chin-Kondner method to determine the bearing capacity using each method separately. Furthermore, these results were compared to the outcome of CAPWAP analysis. The four types of composite piles tested were concrete filled fiberglass shell piles (Lancaster Composite, Inc., pile), polyethylene piles reinforced with steel bars (PPI pile), polyethylene piles reinforced with fiberglass bars (SEAPILE pile) and polyethylene piles (American Ecoboard pile). A summary of the entire process of pile installation and the results between static load and dynamic tests were provided (Robinson & Iskander, 2008).

For composite piles suffering from lateral loading, a test pile program using a statnamic device was conducted and comparisons made between pre-stressed concrete piles and composite piles. It was concluded that the composite pile exhibited a much lower stiffness than the pre-stressed concrete pile. This also illustrated that the composite piles had the ability to sustain lateral load (Pando et al., 2004). To determine the lateral behaviour of large diameter composite piles, tests were conducted by Thomas G. Thomann and Theodore Zoli using two different tests, the inclinometer measurement and survey measurement instead of a statnamic method. It was concluded that the measured deflections were much higher than the pre-load test calculation results. It was also concluded that when using post-load analysis, 85% reduction in soil strength was required (Thomann et al., 2004). Apart from large diameter composite piles, two other types of piles, namely the concrete-filled GFRP pipe pile and the standard pre-stressed concrete pile, were analysed to determine the behaviour lateral loading and the results compared. These indicated that the concrete-filled GFRP pile was more flexible than the standard pre-stressed concrete pile. Also, the ultimate lateral load capacity of the GFRP pipe pile was greater; however, it exhibited brittle behaviour at failure. Additionally, the results concluded that GFRP piles can be modelled using P-y curves and classical beam theory (Weaver et al., 2008). The limitation of all these previous investigations into passive composite piles is that there is no consideration of soil movement during excavation.

In order to overcome the problem of lack of data for passive composite pile behaviour, new research was conducted on a Chinese subway station construction project in western Jinan City, E116° 54'-E117°02', and N36°35'-N36°40'.

2. SITE DISCRIPTION

The construction area was 356.6m×19.7m with an excavation of 16.3m. Concrete bored piles reinforced by GFRP bars were cast at the shield construction side. As shown in Figures 1 and 2, in the soft-eye opening area, these GFRP piles would later be cut by Tunnel Boring Machines (TBM) for tunnelling.

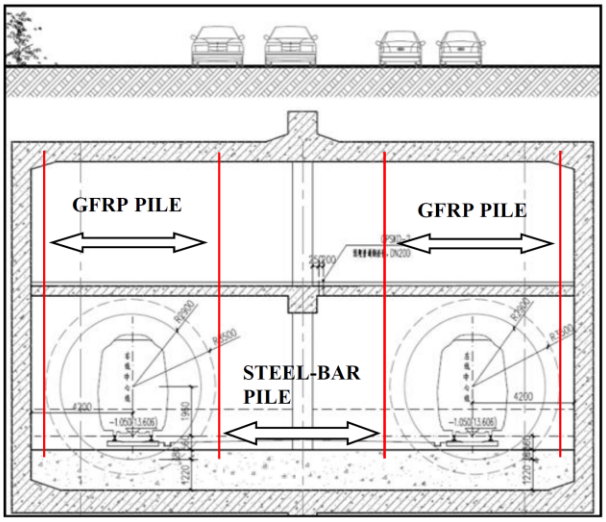


Figure 1 Construction Profile Map (not to scale)

The purpose of replacing the steel bar with GFRP bars is that the GFRP reinforcement allows the designers and contractors to find innovative solutions for the familiar problem of steel reinforced structures which obstruct the TBM head tunnelling through (Schurch & Jost, 2006).

Other piles are made of traditional steel reinforcement with concrete, and all these concrete piles (GFRP & Steel-Reinforced Piles) are connected by concrete top beams as shown in Figure 2, 12, 14 and 15. Three types of piles (Type A, B and C) are used and are shown in Figure 2 with red, blue and yellow coloured blocks. In addition, a water proof wall (Figure 2) was applied to avoid water permeation into the deep excavation pit. Three layers of supports which helped resistance to the changed earth pressure were used in this construction. The construction process was as follows:

1) Concrete supports were installed (Figure 2) (no earth pressure) and later, the excavation started. These passive piles started to resist the loading caused by soil movement. Note that near pile Type C, anchorage was installed;
2) after 6.5 meters of soil was removed, the second steel support was installed, and a further 4.8 meters of soil was removed for the third steel support installation;
3) after the third steel support installation; three additional meters of soil was removed and the total excavation depth of 16.m was achieved.

Because Type A and Type B experienced similar geotechnical conditions with no anchorage application and were made of same material, and because the dimensions of length and diameter were the same as well, the behaviours of these two types of piles (GFRP and steel reinforced passive piles) during soil excavation were researched. As summarized in Table 1, three supports were installed on 18-03-2016, 22-04-2016 and 27-05-2016. Once these supports were installed, the excavation started for 34, 35 and 36 days, respectively. In order to ensure the behaviour of the passive pile remained stable after the last excavation was completed, a further 26 days monitoring was maintained.

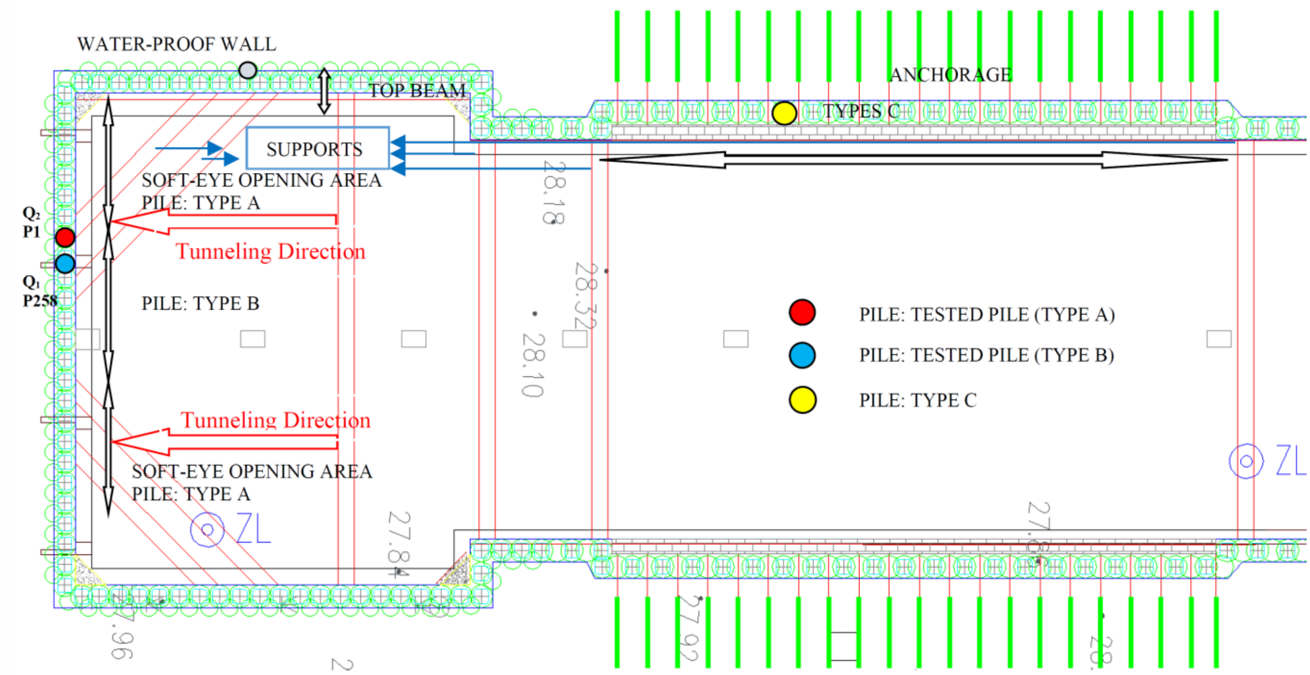


Figure 2 Construction Plane Map (not to scale)

Table 1 Support Installation Information & Excavation Duration

Type of Support	Support Installed Date	Excavation Date	Excavation Duration
Concrete Support	18-03-2016	22-04-2016	34 Days
Steel Support	22-04-2016	27-05-2016	35 Days
Steel Support	27-05-2016	03-07-2016	36 Days
N/A	03-07-2016	29-07-2016	26 Days

3. SUBSURFACE CONDITIONS

Before the projects started, boreholes were driven to depths ranging from 31.0m to 52.0m in order to determine the subsurface. In total, 401 standard penetration tests (SPT) and dynamic penetration tests were conducted, which the later reached the bottom of gravel and medium sand. In addition, the laboratory tests of direct shear test, consolidation shear test and triaxle tests (CU & UU) etc. were conducted. The parameters of subsurface soils were obtained from these tests and are provided in Table 2.

The interpretation of the borehole logs which were near the two tested piles showed that the simplified soil strata were as follows. This is also depicted in Figure 3.

Stratum 1: The first layer is miscellaneous fill which consists of stone, concrete, bricks and soil; the average depth is 2.5m.

Stratum 2: The second layer is yellowish-brown loess; the average depth was 2.9m.

Stratum 3: The third layer is tawny silt; the average depth is 1.2m.

Stratum 4: The fourth layer is silty clay; the average depth is 4.5m.

Stratum 5-1: This layer is dense unsaturated soils with pebble; the pebble diameter ranges from 20mm to 60mm; this only existed near the borehole of YMX037.

Stratum 5: The fifth layer is dark brown medium fine sand; the average depth is 2.6m.

Stratum 6: The sixth layer is dark brown silty clay with average depth of 6m.

Stratum 7: The seventh layer is tawny clay; the average depth is 4.0m.

Table 2 Parameter of Simplified Soil Layers

Soil Layer	Thickness (m)	Average SPT N Value	ω	G_s	ρ	ρ_d	*Es*	*e*	w_L	w_P	I_P	I_L	*c*	φ
			%	-	g/cm³	g/cm³	*MPa*	-	%	%	-	-	*kPa*	°
Fill	2.5	6	N/A	N/A	1.8	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Loess	2.9	13	24.5	2.7	1.93	1.55	9.3	0.75	30.7	18.9	11.8	0.55	29	2 4
Silt	1.2	18	23.2	2.69	1.96	1.59	15.8	0.69	27.5	18.8	8.7	0.51	19	20
Silty Clay	4.5	22	26.4	2.72	1.95	1.54	9.1	0.77	32.5	19.7	12.8	0.53	36	22
Fine Sand	2.6	26	N/A	N/A	1.96	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0	22
Silty Clay	6	29	27.3	2.72	1.93	1.52	7.2	0.8	34.3	20.7	13.8	0.47	42	16
Clay	4.0	33	28.6	2.75	1.93	1.51	6.9	0.793	43.5	23.5	20	0.3	38	15

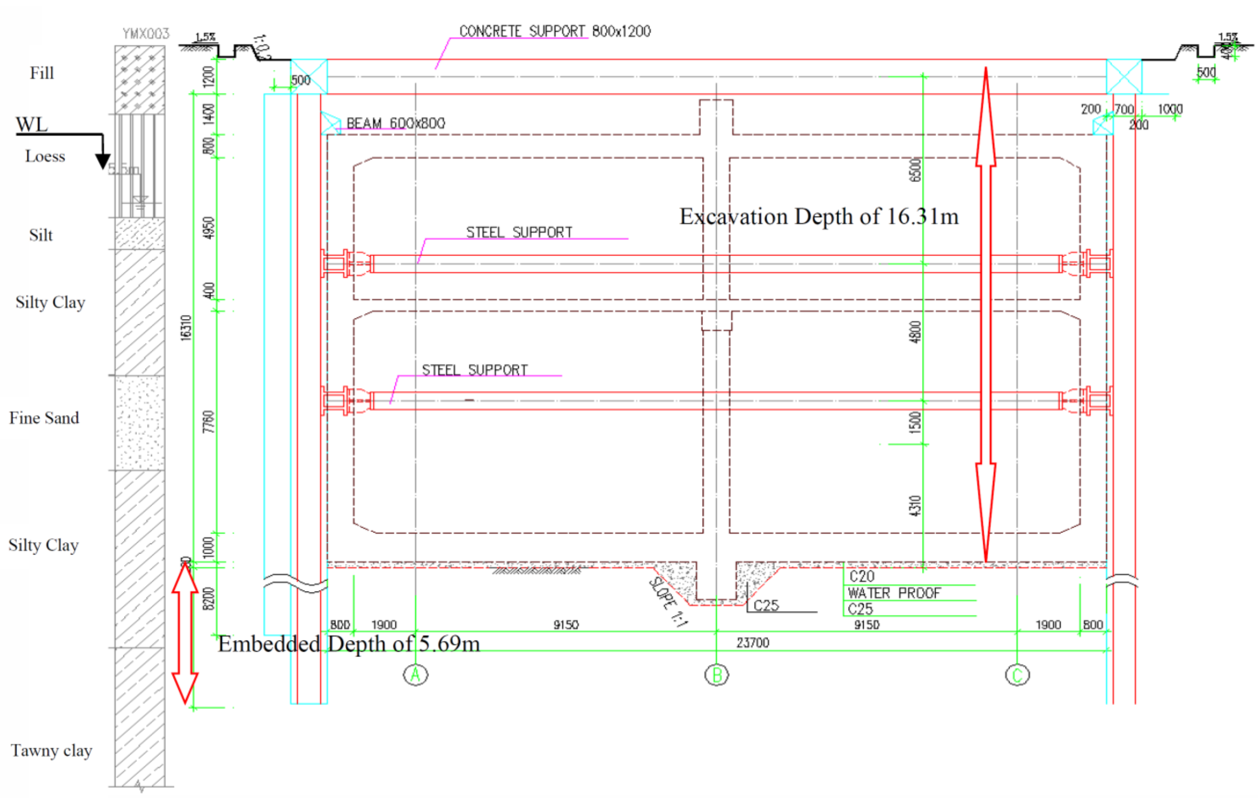


Figure 3 Soil Layers adjacent to Pile (not to scale)

4. PILE CHARACTERICS

The manufacture process of GFRP bar reinforced concrete piles and steel bar reinforced concrete piles was driving holes, assembling reinforcement, and transferring into the holes and casting. The monitored piles which suffered from lateral loading were GFRP bar and steel bar reinforced concrete piles, respectively. These two bored piles were all designed to be 22m long with pile diameters of 700mm. The GFRP stirrups were 10mm in diameter with spacing of 150mm (Figure 4); the longitudinal reinforcement was 26 GFRP bars with diameters of 28mm (Figure 5). The ultimate tensile strength of the GFRP bar was over 500MPa and the designed grade of the concrete was C30. Based on the Chinese Concrete Code GB50010-2010, the cubic compression strength of C30 was 30MPa (dimensions of the cubic concrete sample are 150mm×150mm×150mm). The GFRP cage is assembled as shown in Figure 6.

Figure 4 GFRP Stirrups of Piles

Figure 5 GFRP Reinforcements of Piles

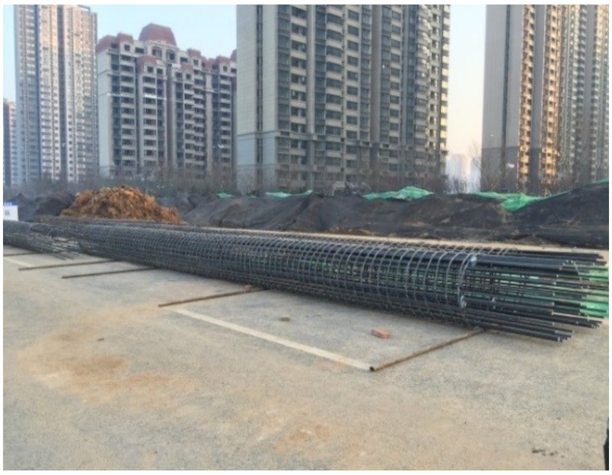


Figure 6 Assembled GFRP Cage of Piles

The traditionally steel reinforced concrete pile was also made of C30 concrete, the stirrups were 16mm diameter HPB300 (Hot Rolled Plan Bar) bars with spacing of 150mm and the horizontal reinforcement was 26 HRB400 (Hot Rolled Ribbed Bar) bars with diameters of 28mm. The ultimate tensile strength of the concrete was 30MPa, and the yield strength of HPB300 and HRB400 were 300MPa and 400Mpa, respectively. The steel cages were welded and the finished steel cages can be seen on the left-hand side of Figure 7. The parameters of GFRP and steel reinforcement are summarized in Table 3.

Figure 7 Assembled Steel Cage of Pile

Table 3 Parameters and Price of Reinforcement

Types of Pile	GFRP Pile		Steel Pile	
Reinforcement	Horizontal reinforcement	Stirrup	Horizontal reinforcement	Stirrup
Brand	GFRP500	GFRP bar	HRB400	HPB300
Diameter	28mm	26mm	26mm	26mm
Yield Strength	500MPa	300MPa	400Mpa	300Mpa
price	18.5￥/m	18.5￥/m	4.8￥/m	4.8￥/m

The reason for utilizing a GFRP pile as substitute reinforcement is that GFRP is a brittle material and ductility is comparatively small. As shown in Figure 2, for the purpose of breaking through the soft-eye opening area, the GFRP pile, which will suffer from brittle failure, is preferred because if traditional reinforced concrete piles are used, the reinforcing steel cannot be easily cut through by the TBM, and consequently unsafe manual labour is needed to remove the reinforcing steel.

This GFRP material is non-corrodible with long term durability and possesses high tensile strength; except the cost is very high. As illustrated in Table 3, the cost of GFRP material is approximately four times that of the steel reinforcement. If the substitution of all steel reinforced concrete piles with GFRP reinforced concrete piles was applied, this would cost the project (516 piles, all with lengths of over 20 meters) 3,096,000￥ (595,384.7AU$) or more. Note that, in this project design, the tensile strength of this GFRP was deliberately divided by high safety coefficients due to its rare usage, . Utilization of an appropriate design, therefore, needs continued research.

5. TEST SETUP AND PROCEDURES

The measurement system for determining the lateral deflection was plastic PVC tubes and a readout probe. The internal diameter of the PVC tube was 75mm with 4 grooves cut at 90 degree intervals. Figures 8 and 9 illustrate the installation of PVC tubes on GFRP cages and steel cages, respectively. The readout probe that fits into the grooves is shown in Figure 10.

Figure 8 PVC Tube on GFRP Cage

Figure 9 PVC Tube on Steel Cage

The amount and location of horizontal movement in a deep foundation can be determined through lowering down the readout probe into the bottom of PVC tube and pulling it upwards. During system installation, the top of the PVC was covered by a cap to avoid the concrete being pulled inside, and it was then covered again by a woven bag in case the plastic cap was damaged by coarse aggregate.

Figure 10 Readout Probe for Deflection Measurement of Pile

After the reinforcement cages were assembled and the PVC tubes were installed, the cages were then transferred to the required location. The cage of each pile was lifted by two cranes and was then transferred into the guide slot. In order to make sure the concrete cover met the requirements, concrete cushion blocks were used with spacing of 2m. Figure 11 shows the installation of the steel reinforced concrete bored passive pile. After the installation of the GFRP and steel reinforcements, the fresh concrete was poured into the holes.

The observation of pile behaviours was conducted in the soft-eye opening area for safety purposes during shield construction. As shown in Figure 2, the steel bar and GFRP bar reinforced concrete piles were located at tabs of P258 and P1, respectively. Each two piles were 1.8m distance apart so the geotechnical condition was believed to be the same. The displacement measurement labels were Q_1 and Q_2. During the excavation, three layers of supports were used; the first layer of supports was made of concrete and the other two were steel supports. Note that these two monitored piles were not supported by any supports.

Figure 11 Installation of Steel Reinforced Concrete Pile

After the concrete piles were cured, the lateral deflection measurement was started before the concrete support was cast and data was recorded as initial displacement.

The readout probe was firstly lowered down to the bottom of the PVC tube and lifted up 0.5m after recording the initial data of A_0 (the data are labelled as A_1, A_2, $A_3 \ldots A_j$). Once reaching the pile head, the probe was rotated and lowered down again, and then pulled up again to record the data of B_{180}, which represents the data after the rotation of 180 degrees (the data are labelled as B_1, B_2, $B_3 \ldots B_j$).

The angular deflection of the tube, and amount and location of horizontal movement in the pile foundation can be determined and the accumulated horizontal displacement is then obtained by Eq. (1).

$$X_i = \sum_{j=0}^{i} L \sin \alpha_j = C \sum_{j=0}^{i} (A_j - B_j) \quad (1)$$

Where:
A_j: Collected data of time i with respect of 0^o
B_j: Collected data of time i with respect of 180^o
C: Constant referring to the equipment
L: Observing depth each time, 500mm
α : Pile angle due to earth pressure.

Because of the installation of concrete support in which ultimate compression strain is very small, assuming that the pile head should not move (if pile head moves, the concrete support will break), the accumulated deflection value changes from pile head to the pile toe. The lateral displacement measurement was recorded every day after all three layers of supports were installed and the soil excavation finished.

6. RESULTS OF GFRP & STEEL REINFORCED CONCRETE PILES

The concrete supports were cast before excavation. As shown in Figure 12, the excavator dug out soil after the concrete support was cured (cured date: 18-03-2016). It was believed that there was no lateral displacement of the pile at this stage since the earth pressure was relatively small during the first excavation. However, there were three excavators, which created extra pressure to push the pile head outside the excavation site.

Figure 12 Excavation after First Concrete Support was installed

The obtained lateral displacement of the upper part of the GFRP pile was discovered within +0.5mm (negative value represents load direction points to foundation pit), possibly caused by the excavators. This small value could also have been caused by the accuracy of the inclinometer.

Under the assumption of 0 movements of the pile head, the accumulated displacement values were overestimated by calculation especially from the pile bottom. As shown in Figure 13, the data shows that, during the five days of excavation from 18-03-2016 to 23-03-2016, lateral deflection was detected within +0.4mm from the pile head (error due to equipment or effect by the excavators), and the accumulated lateral deflection was +0.73mm at the bottom of pile, which is calculated by adding error values from the pile top.

As the excavation kept going, the earth pressure started to increase, which meant that the data could be captured by the equipment accurately. As shown in Figure 13, after 01-04-2016, the recorded data could be trusted because negative value occurred, which represented that the earth pressure had pushed the pile to the excavation pit.

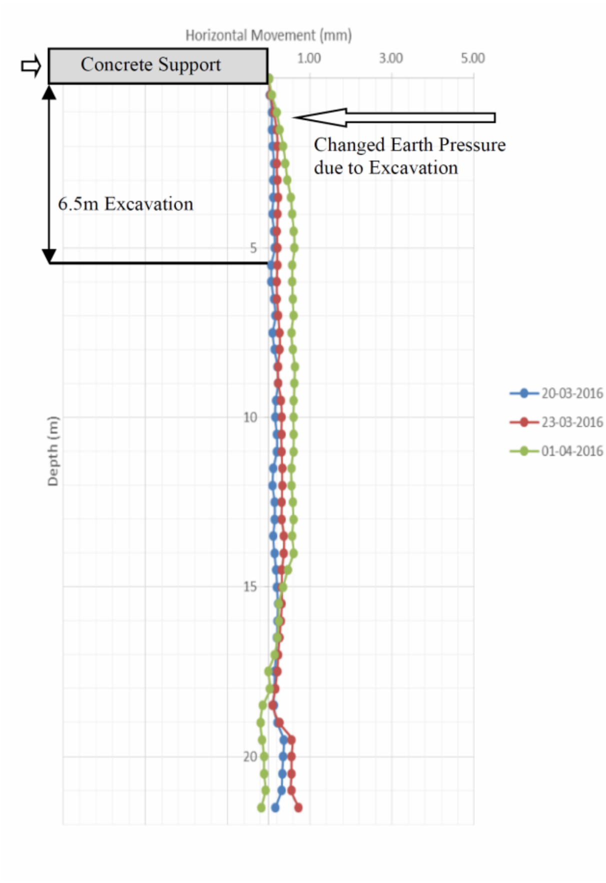


Figure 13 Deflection of CFRP Pile at the beginning of Excavation

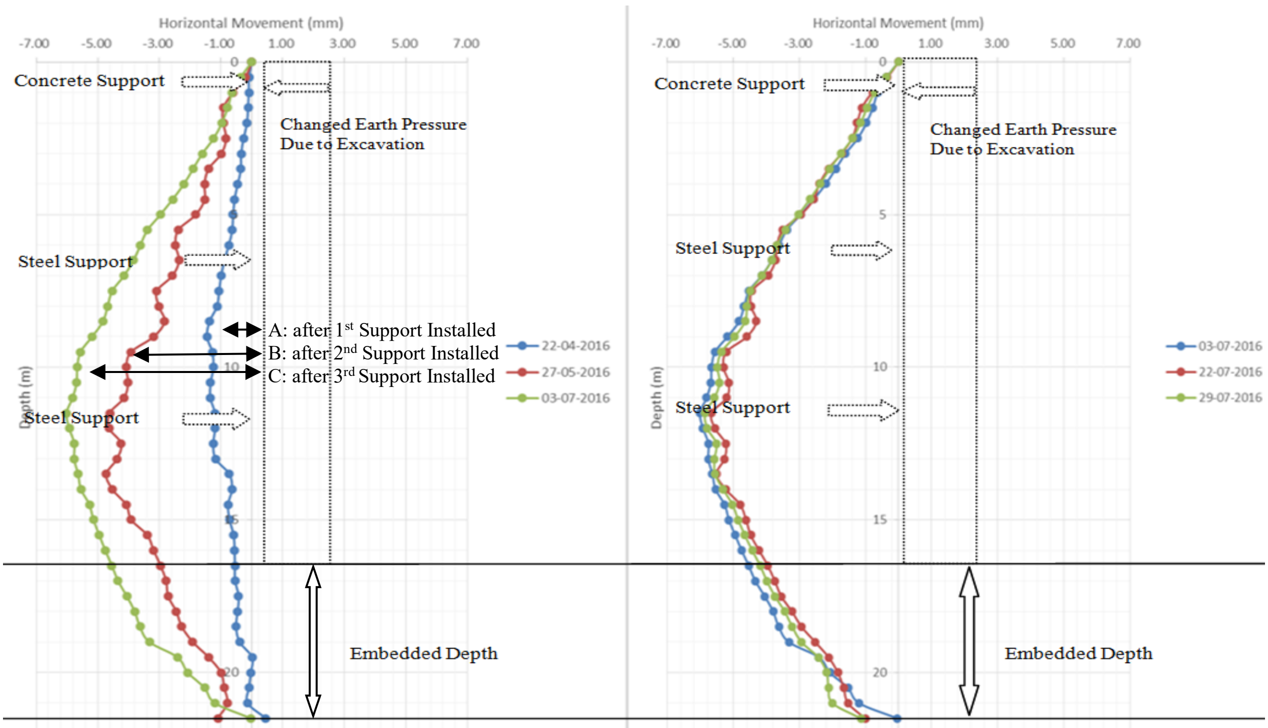
The first and second steel supports were installed on 22-04-2016 and 27-05-2016, respectively as shown in Figures 14 and 15. After these three supports were installed, the deflection behaviours of the GFRP reinforced concrete pile were recorded as shown in Figure 16 (a) with labels a, b, and c representing the collected data after installation of the three types of supports. As demonstrated in Figure 16 (b), during 26 days of continuous data collection, the horizontal movement remained stable.

Figure 14 First Steel Support Applications

Figure 15 Second Steel Support Applications

Through analysis of the data obtained from the inclinometer (Figure 16, (a)), it was detected that lateral deflection of FRP reinforced concrete pile was increasing as excavation depth was increasing. It was detected that the maximum horizontal deflection was -1.5mm at the depth of 13m when the second support was installed (the first steel support). After the first and second steel supports were installed, the maximum deflections were discovered at the depth of 12.5m and 13m, with values of -4.7mm and -5.8mm, respectively. Additionally, after the last steel support was installed, the data collection kept going on. The deflection became stable after 03-07-2016 (Figure 15(b)). It was also detected that the horizontal movement of the pile toe was stable at the value of –1.1mm, which led to the heave in the excavation pit (also discovered by total station). This phenomenon proves the correct assumption of 0 movement from the pile head.

Simultaneously, the steel reinforced concrete pile was also monitored. After approximately 15 weeks of monitoring, the lateral deflection data of the traditional steel reinforced concrete pile obtained from the inclinometer system demonstrated that, after the concrete support and two steel supports were installed, the maximum lateral deflections were –4.70mm (at the depth of 12.5m), -7.95mm (at the depth of 12.5m) and -10.56mm (at the depth of 12.5m), respectively as shown in Figure 17. Comparatively, the deflection behaviour of these two piles is summarized in Table 4.

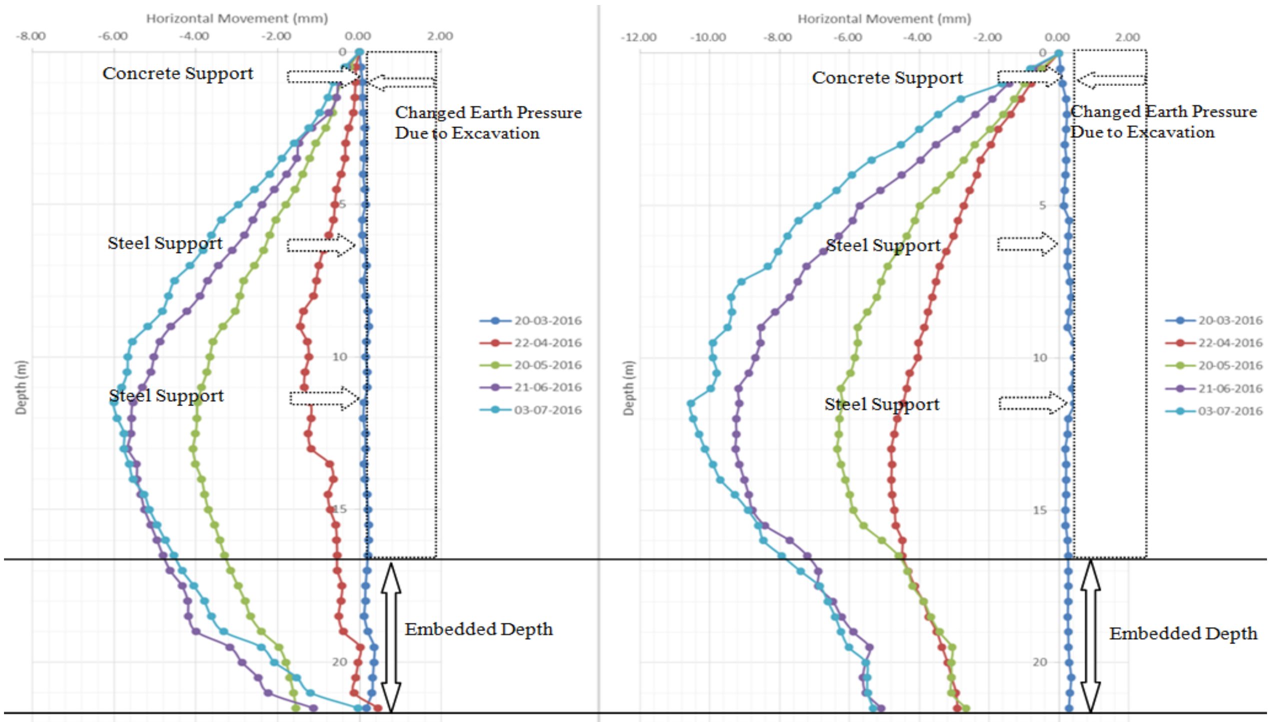


(a) (b)

Figure 16 (a) Lateral Deflection of GFRP Pile during 3 Stages; (b) Total Horizontal Deflection of FRP Pile

Table 4 The Deflection Behaviors of CFRP and Steel Reinforced Concrete Piles

	GFRP Reinforced Concrete Pile		Steel Reinforced Concrete Pile	
Construction Condition (After)	Max. Deflection (mm)	Depth of (m)	Max. Deflection (mm)	Depth of (m)
First Concrete Support Installed	-1.56mm	12.0m	-4.70mm	12.5m
First Steel Support Installed	-4.73mm	12.5m	-7.95mm	12.5m
Second Steel Support Installed	-6.02mm	12.5m	-10.56mm	12.5m



(a) (b)

Figure 17 Deflection of GFRP Bar (a) and Steel Bar (b) Reinforced Concrete Bored Piles

7. CONCLUSIONS

Analysis of the lateral displacement diagrams of the piles evidences that the deflection is increasing along the pile length with no turning point. It can then be concluded that the concrete and steel supports made little contribution and the project is, therefore, over-designed. Lateral deflection tests between GFRP reinforced concrete piles and steel reinforced concrete piles were conducted and the installation procedure of piles was provided. The GFRP composite pile demonstrated maximum lateral deflection of -6.02mm at the depth of -10.56m and the maximum lateral deflection of steel reinforced concrete bored pile was -9.02mm at the depth of 13.5m.

It cannot be concluded that the GFRP bar reinforced concrete pile demonstrated better than the traditionally reinforced concrete pile since the ultimate tensile strength of GFRP and steel are different (GFRP>500MPa; Steel=400MPa). However, it can be concluded that the GFRP bar reinforced concrete bored pile can be a suitable alternative to replace the traditionally reinforced concrete bored pile in deep excavation.

It is noteworthy that there is limited structural analysis of passive piles with GFRP reinforcement since the purpose of its utilization is its brittle property which avoids damage of TBM. In addition, there is no research associated with passive piles with FRP laminar outside reinforcement applications. There are some tests which have been conducted for the FRP composite piles which suffer from vertical loading; however, these are all focused on the FRP tubes (concrete-filled FRP pile; FRP tube pile; structurally reinforced composite pile, etc.) and plastic piles with fibre (e.g. American Ecoboard pile). To the best of our knowledge, there is no field test which refers to the piles confined by FRP laminar suffering from vertical loading and no research associated with the fibre orientation.

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