Planning and Interpretation of Instrumented Lateral Pile Test Performance with a Semi Restrained Pile Head Condition

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ABSTRACT: This paper describes the planning of the instrumentation scheme in a hollow circular prestressed spun pile with an intended structural frame attached to pile head for a fixed head condition. The installation detailing for the inclinometer inside the prestressed spun pile is described. Due to high free standing length of the test pile over water in order to avoid any adverse wave action during the test but resulting in high induced moment from the lateral test load, this testing arrangement has made the interpretation of the test results very difficult and challenging to yield the useful representative test performance. The interpretation of the pile internal stresses before the cracking of concrete, particularly the pile flexural stress, is performed using linear elastic beam theory with the pile deflection profile for the upper free standing pile segment as a free body to produce stiffness response for the soil embedded pile shaft. The pile head connection fixity is also discussed and quantified theoretically for comparison.

KEYWORDS: Lateral pile test, Instrumentation, Pile head connection fixity, Pile flexural stress

1. INTRODUCTION

The main objective of performing the full scale lateral pile load test is to verify the performance of the bridge pier foundation piles under lateral loading condition. This paper focuses solely on the technical interpretation of the static lateral load test. Due to high free standing length of the test pile over water to avoid any adverse wave action during the test and the resulted high induced moment from the lateral test load, this testing arrangement has made the interpretation of the test results very difficult and challenging to yield the useful representative test performance.

Along the way of interpreting the test results, the review of lateral pile stiffness, pile structural capacity and also lateral soil capacity and stiffness below riverbed has also been performed. This interpretation can serve to verify the design assumptions on prediction of lateral pile group movement under service loading condition (Serviceability Limit State Check).

The subsoil condition of the site is generally underlain by Quaternary Alluvium. Alluvium deposits at this area generally consist of marine deposits with mainly dominant of sand and gravel, but also consists of clay and silt. The test pile is of 38m long with the details as summarised in Table 1.

Table 1 Test pile details

Diameter of Test Pile	1000mm
Wall Thickness of Test Pile	140mm
Concrete Grade	80
Prestressing Load	7.0N/mm^2
Cracking Bending Moment	1,060 kN-m/pile
Ultimate Bending Moment	1,597 kN-m/pile
Elastic Modulus, E	40GPa
Moment of Inertia, $I_{uncracked}$	3.59 x 10^{10} mm^4
Flexural Stiffness, $EI_{uncracked}$	1,436,000 kNm^2

2. PILE INSTALLATION AND INSTRUMENTATION

The test pile was installed using 35 tons hydraulic impact hammer at 800mm drop height. The test pile was terminated at penetration depth of 17.4m from riverbed with a total pile length of 38.0m. The pile cut-off level is located at 12.5m below the pile head and the riverbed level is at 20.4m below pile head.

In order to impose lateral test load to the test pile, a hydraulic jack with calibrated load cell for primary load measurement was placed at 4m below the pile head and jacked against the working platform supported by 5 numbers of 1000mm diameter steel pipe piles with sufficient embedment into the riverbed for deriving the necessary testing reaction load as shown in Figure 1.

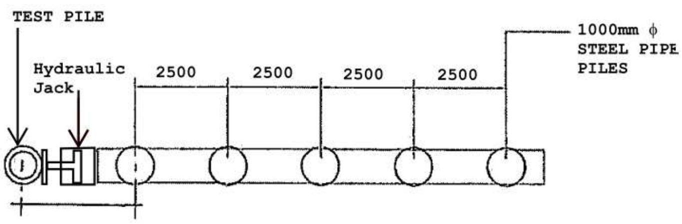

Figure 1 Lateral reaction load setting out

There was also an attempt to minimise the pile head rotation with some counter moment from the reaction frame in order to reduce the flexural stress at the test pile body when subjecting to laterally imposed test load. Hence an A-frame truss with roller supports sliding laterally over the working platform was fixed onto the test pile head at two points (Points T and B) to restrain the pile head rotation while allowing free lateral movement as shown in Figure 2.

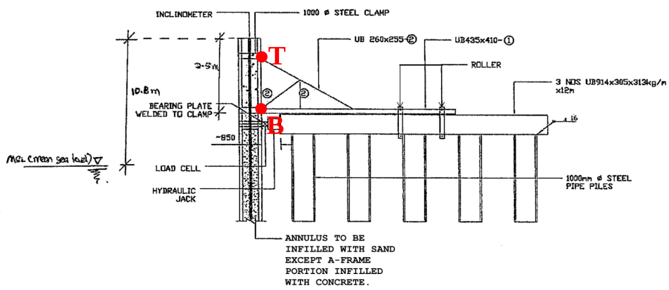

Figure 2 Lateral load test setting up

Linear variable differential transformers (LVDT) S1 and S2 were attached on a separate reference column to measure the lateral pile head deflection and the other two LVDT transducers (S3 and S4) were attached onto the working platform to measure the vertical and lateral movements of the A-frame truss respectively. The actual installed locations of the LVDTs are summarised in Table 2 and shown in Figure 3.

Table 2 Location of LVDT

LVDT	Installed Location
S1	3m below pile top
S2	2m below pile top
S3	At reaction piles
S4	At reaction piles

However, due to the practicality of testing over maritime environment, the testing setup with safety consideration had to be constructed on a higher working platform to avoid the influence of unfavourable wave action from the tides, thus the location of the lateral load imposition was inevitably raised to higher level (8m higher than actual cut-off level) that will introduce extra flexural stress to the test pile corresponding to the imposed lateral load in the actual condition. This extra bending effect due to safe testing setup shall be duly accounted for during interpreting the test results as it will make the test pile attaining flexural structural failure earlier before full mobilisation of the geotechnical pile capacity of the foundation subsoil embedding the test pile.

In addition to the above instrumentation on load and deflection measurements, the entire test pile deflection profile at each loading interval was also taken from the inclinometer embedded in the pile centre annulus to the full pile length with sand backfill as shown in Figure 3.

The reason of using sand backfill is to avoid altering the pile flexural stiffness with the use of cementitious infill traditionally, that lead to unnecessary structural stiffening effect. With the pile deflection profile available at every loading condition, the pile deflection profile, deflection curvature and inclination can be derived for interpretation purpose.

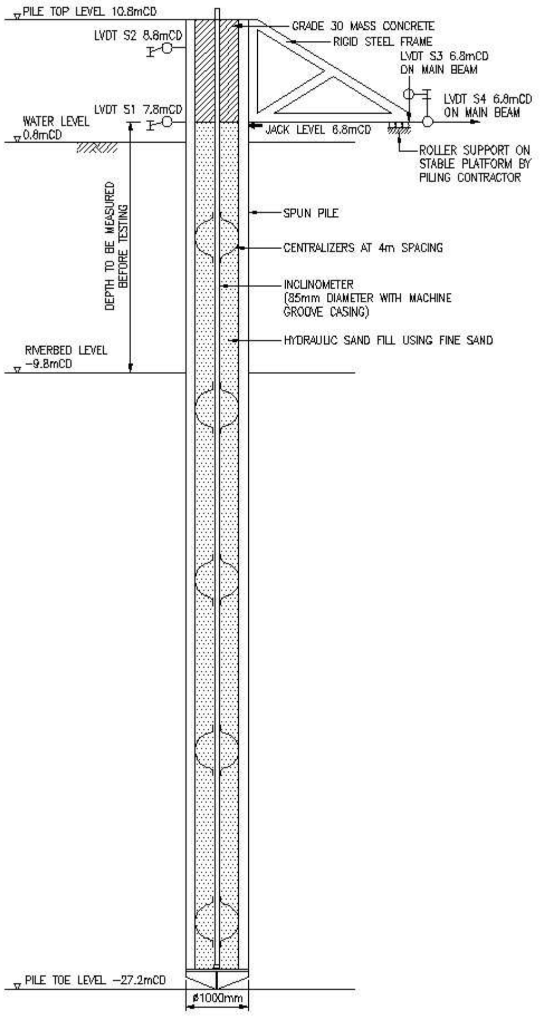

Figure 3 Details of instrumented lateral pile load test

3. LATERAL PILE LOAD TEST RESULTS

LVDT S1 and S2 show similar behaviour after the applied lateral load. For easy reference, only lateral movement at LVDT S1 readings is referred in the subsequent discussions in this paper. In the first loading cycle, the observed maximum lateral pile deflection at the peak load of 137.8kN was 147.53mm. Upon unloading to zero, the pile rebounded to a residual lateral deflection of 18.32mm.

In the second loading cycle, the observed maximum lateral pile deflection at the peak load of 220.0kN was 398.67mm for the test pile. The test was terminated at this stage due to excessive lateral movement causing inclinometer probe jammed in the test pile. The result of LVDT S1 is shown in the Figure 4.

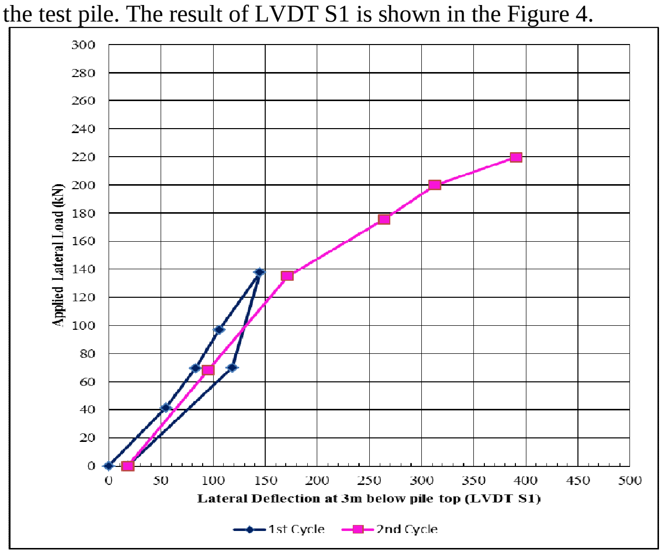

Figure 4 Applied Lateral Loads (kN) versus Lateral Deflection (mm) at 3m below Pile Top (LVDT S1) for Lateral Pile Test

From the inclinometer results during the loading stage of the first cycle as shown in Figure 5, it can be observed that the upper part of the pile undergoes significant displacement and the effects of the applied lateral load diminishes at some depth down of the pile. However, during the loading stage of the 2nd cycle, the inclinometer results at the 165% of lateral pile working load (test load of 220.0kN) shows abnormal behaviour in which the actual deflection cannot be correctly recorded as shown in Figure 6.

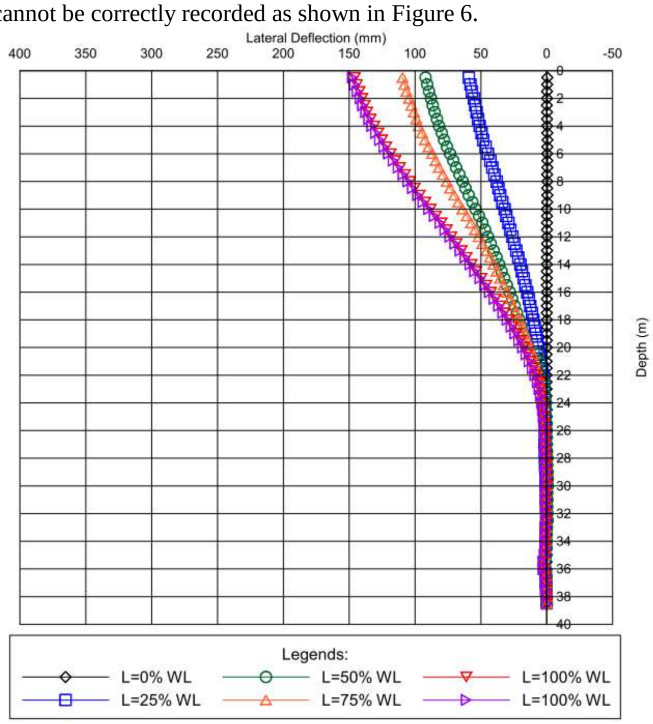

Figure 5 Lateral deflection (mm) profiles from inclinometer reading during loading stages (1st Cycle)

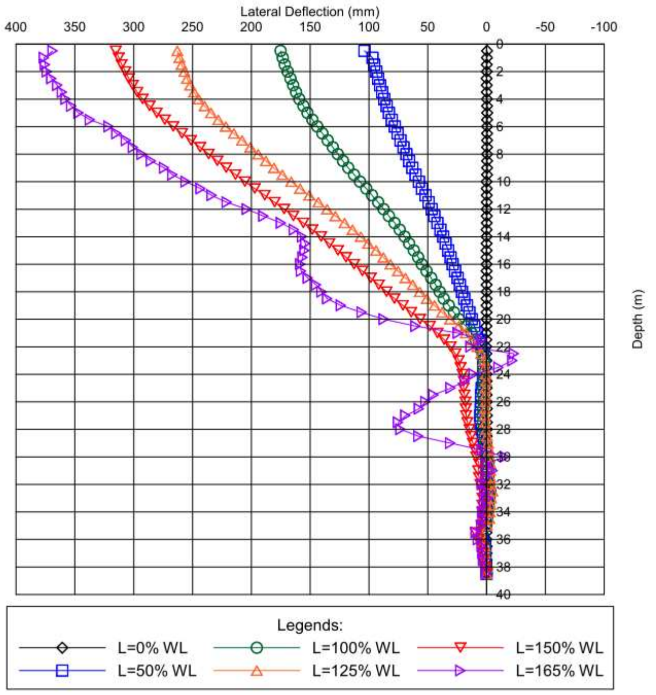

Figure 6 Lateral deflection (mm) profiles from inclinometer reading during loading stages (2nd Cycle)

4. INTERPRETATION OF LATERAL PILE LOAD TEST RESULTS

Since the pile bending moment and lateral load in this test pile will not be in tandem with the coupled loading conditions of the actual pile foundation loading in the design, it is important to calibrate both the structural model and geotechnical model with the deviated load combination of lateral load and the extra high induced moment. Thereafter, to reapply the calibrated model to compute the expected pile performance behaviour based on the anticipated load combination. The factor of safety for the pile structural and geotechnical capacity shall be verified under the ultimate limit state condition as identified in the interpreted test results as well.

4.1 Structural Pile Capacity Assessment

As there is no any external load acting along the pile body between the jack and riverbed other than the jack load and the reactions below the riverbed, it would be useful to examine the structural behaviour of this structural portion of the test pile for its structural stiffness performance and capacity using simple linear elastic beam with end moment restraints.

From the observation of the pile deflection profile, it appears that the pile head restraint is not fully fixed or restrained as the actual restraints of counter moment by a force coupler at point T and B only, but is considerably rigid as the maximum pile head rotation is 0.3 degrees when the lateral working load of 135kN was reached. However, it will be theoretically correct to extend a tangent line of the deflected pile profile at the riverbed and also another tangent line at the jack position as shown in the sketch in Figure 7 to compute the induced end moments at both ends taking a free body of pile portion between the riverbed and the jack position.

With the two tangent lines mentioned earlier and the pile deflection profile, then the end moments of the pile free body at the jack level and riverbed can be derived using linear elastic beam theory with end restraining moments. Assuming the pile flexural stiffness still remains constant as uncracked section for the initial loading stage until the starting of deterioration of pile stiffness indicating concrete cracking to fully develop plastic hinges implying the test pile approaching ultimate limit state structurally, the derivation of the end moments at the jack level and riverbed using linear elastic beam theory before cracking of concrete shall remain valid. The summary of the linear elastic pile deflection analysis is tabulated in Table 3 and Table 4.

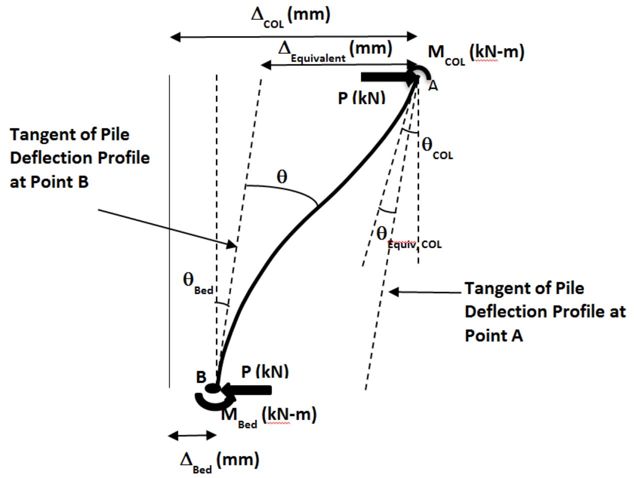

Figure 7 Sketch of computed structural flexural analysis of freestanding length of laterally loaded test pile

Table 3 Summary of linear pile deflection analysis for first cycle

Load, P (kN)	Load, P (% of WL)	Deflection at Jacked Level, Δ_{Jacked} (mm)	Deflection at River Bed Level, Δ_{Bed} (mm)	Rotation at River Bed Level, θ_{Bed} (rad)	Deflection, $\Delta_{Equivalent}$ (mm)
0	0	-0.3	0.1		-0.4
41	31	50.9	5.3	0.0021	10.3
70	51	75.05	5.7	0.0022	33.4
97	72	97.2	10.8	0.0041	18.1
138	102	132.9	15.9	0.0053	29.6
70	52	109.5	14.4	0.0040	28.2
0	0	17.8	3.7		14.1

Table 4 Summary of linear pile deflection analysis for second cycle

Load, P (kN)	Load, P (% of WL)	Deflection at Jacked Level, Δ_{Jacked} (mm)	Deflection at River Bed Level, Δ_{Bed} (mm)	Rotation at River Bed Level, θ_{Bed} (rad)	Deflection, $\Delta_{Equivalent}$ (mm)
68	50	87	10.4	0.0030	27.6
135	100	159.2	17.5	0.0068	28.1
176	130	244.8	23.4	0.0090	71.2
200	148	292.3	48.2	0.0102	74.2
220	163	357.8	60.3		297.5

The same behavior is also shown in the tabulation in Table 5 where the rigidity factor, α, is fairly close to 1.0 implying fixed head condition with pile head rotation from the tangent line drawn at the river bed level of the pile deflection profile and the load up to maximum test of 138kN in the first cycle. Rigidity factor is tabulated from Equation (1).

$$\alpha = 4/3 - 4EI\theta/(PL^2) \quad (1)$$

Where EI is the flexural thickness and P is the testing load. L is the pile length below the river bed, which is 16.6m in this case. Equivalent rotation, $\theta_{Equivalent}$ in radian shall be derived from the equivalent deflection, $\Delta_{Equivalent}$.

Table 5: Summary of rigidity factor, α

	Load, P (kN)	Load, P (% of WL)	Equivalent Deflection, $\Delta_{Equivalent}$ (mm)	Equivalent Rotation, $\theta_{Equivalent}$ (Rad)	Rigidity Factor, α
1st Load Cycle	0	0	-0.4	0.000	**N/A**
	41	31	10.3	0.001	1.020
	70	51	33.4	0.002	0.728
	97	72	18.1	0.001	1.098
	138	102	29.6	0.002	1.063
	70	52	28.2	0.002	0.825
	0	0	14.1	0.001	**N/A**
2nd Load Cycle	68	50	27.6	0.002	0.824
	135	100	28.1	0.002	1.071
	176	130	71.2	0.004	0.822
	200	148	74.2	0.004	0.866
	220	163	297.5	0.018	**N/A**

It is significant to back-calculated the restraining moment induced by the lateral test load at the jacked level and riverbed to understand the force experienced by the pile. The moments at jacked level and at river bed are derived by using Equation (2).

$$M = \alpha PL/2 \quad (2)$$

Where M is the derived restraining moment and P is the imposed lateral test load. L is the pile length between river bed and jack, which is 16.6m in this case and α is the rigidity factor tabulated in Table 5. Table 6 and Table 7 show the summary of derived restraining moments for first cycle and second cycle respectively.

Table 6: Summary of derived restraining moments for first cycle

Applied Load, P (kN)	Applied Load, P (% of WL)	Rigidity Factor, α	M_{Jack} (kN-m)	M_{Bed} (kN-m)
0	0	**N/A**	N/A	N/A
41	31	1.020	350	336
70	51	0.728	420	733
97	72	1.098	883	725
138	102	1.063	1216	1072
70	52	0.825	480	682
0	0	**N/A**	N/A	N/A

Table 7: Summary of derived restraining moments for second cycle

Applied Load, P (kN)	Applied Load, P (% of WL)	Rigidity Factor, α	M_{Jack} (kN-m)	M_{Bed} (kN-m)
68	50	0.824	466	665
135	100	1.071	1201	1041
176	130	0.822	1198	1715
200	148	0.866	1437	1883
220	163	**N/A**	N/A	N/A

From Figure 8 (Pile Lateral Response at Riverbed), it is also clearly shown that the test pile demonstrates linear elastic flexural behaviour in the pile-soil system with the imposed lateral load until 138kN in the first load cycle. It is also interesting to note that the derived restraining moment (1,216kN-m) at the jack level as tabulated in Table 6 has also just slightly exceeded the pile cracking moment of about 1,060kN-m. Similarly, the derived restraining moment (1,296kN-m) of the pile at riverbed level is also reaching the cracked moment capacity of the test pile. The pile body moment can be determine from actual pile deflections profile measured from the inclinometer for 25%, 75% and 100% load at first cycle at riverbed level. The linear elastic behaviour of test pile with lateral load up to 138kN without observing development of any flexural hinge along the pile shaft and the derived end restraining moment of 1,216kN-m at jack level and 1,296kN-m at riverbed supports that the cracking moment can be slightly higher than 1,060kN-m. But for the purpose of subsequent assessment, conservative cracking moment of 1,060kN-m is adopted.

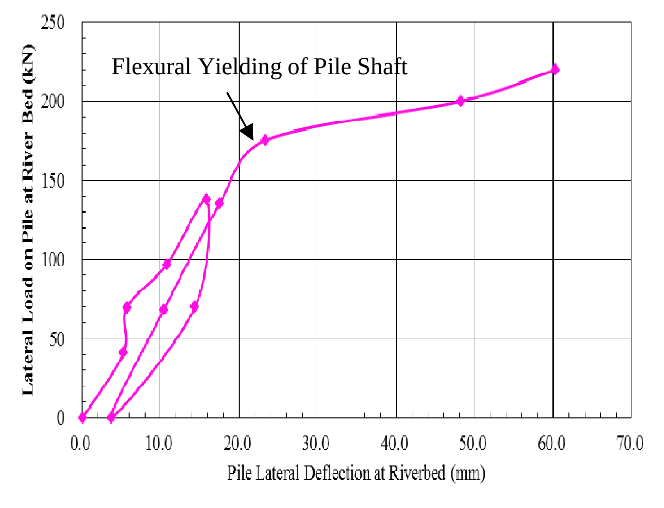

Figure 8 Pile lateral responses at riverbed

Since the cracked moment has reached, the test pile will behave as cracked section under further bending thereafter and cracked section stiffness (which is generally reduced rapidly with the increasing moment imposed on the pile) shall be used to assess subsequent loading cycle.

When the lateral load at the riverbed is increasing from 176kN to 220kN in second load cycle, non-linear yielding behaviour of the pile deflection can be observed. It is believed that the test pile shall have experienced flexural yielding slightly before or either slightly after reaching lateral load of 176kN in the second loading cycle and reached ultimate flexural capacity of the pile at lateral load of 220kN in the second loading cycle. As the test pile has developed two yielding end restraining moments at the close proximity to jack level and riverbed in the second load cycle, the linear elastic flexural analysis will not be meaningful for subsequent loading thereafter.

4.2 Geotechnical Pile Capacity Assessments

For the lateral geotechnical capacity of the pile, the riverbed materials embedding the test pile shall refer to the borehole BH-7 as shown in Figure 9, which is nearest to the test pile. The underlying stratum subsoil is firstly a layer of gravelly silty sand (refers as Zone 1 material) of about 8.8m thick followed by a sandy clay layer (refer as Zone 2 material) of about 6m thick before hitting again the gravelly sandy silt and gravelly clayey silt over the schist bedrock. The uppermost portion of the test pile is mostly embedded by the gravelly silty sand, in which the passive resistance is expected to be high.

Presumably, very conservative strength parameters (ϕ'=28, c'= 0) and the submerged soil density of 8kN/m^3 for the pile embedment materials (Zone 1 material) are considered, the derived geotechnical lateral pile capacity is very high and geotechnical failure (i.e. failing the embedded soil in passive failure mode) is remote with such low lateral working load on the pile. It is not difficult to observe from the pile deflection profile that the lateral movement zone (i.e. critical pile length, l_c) of the test pile in the first loading cycle is within the top 7m below the riverbed, which is fully within the Zone 1 material.

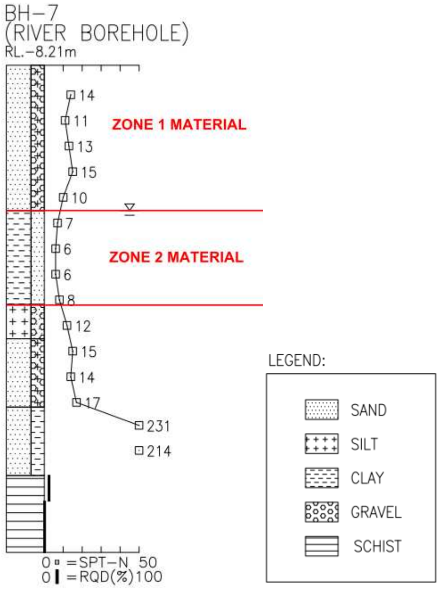

Figure 9 Subsoil of borehole BH-7

According to Fleming (2009), limiting force per unit length of pile, P could be matched sufficiently by the simple expression given below

$$P = K_p^2 \sigma_v' d \tag{3}$$

Where K_p is the passive earth pressure coefficient, equal to (1+sin ϕ')/ (1-sinϕ') and d is the diameter of pile.

The ultimate lateral pile geotechnical capacity of test pile with a conservative assumption of free pile head connection and conservative soil strength parameters is about 426.5kN (if the pile body moment is limited to cracking moment of 1,060kN-m) and 560.5kN (if the pile body moment is limited to ultimate moment of 1,597kN-m) as summarised in Table 8. The geotechnical lateral pile capacity was then compared with the lateral pile working load as the factor of safety (FOS) for installed pile.

Table 8 Summary of the geotechnical lateral pile capacity

Condition	**Limit of Pile Moment Capacity (kNm)**	**Geotechnical Lateral Pile Capacity (kN)**	**Lateral Pile Working Load (kN)**	**FOS (≥ 2.0)**
Cracking of Concrete	1060.0	426.5	135.0	3.2
Ultimate Limit State	1597.0	560.5	135.0	4.2

The factor of safety of geotechnical pile lateral capacity can range from 3.2 to 4.2, which is much higher than the normal requirement of 2.0 (with verification of lateral pile load test) or 3.0 (without verification by lateral pile load test). Hence, the geotechnical capacity of the test pile is of no concern based on the presented interpretation of this lateral load test results.

To calibrate the deformation parameters of the embedded subsoil materials, Table 9 and Table 10 show the back-calculated the soil stiffness profile embedding the test pile by using the computer program, PIGLET in order to fit the pile deflection at riverbed level under the loading combination of the lateral load and moment induced on the pile at riverbed level. PIGLET is the computer software developed by M.F. Randolph (2004) for the purpose of analysis and design of the pile group based on subsoil interaction between soil and pile group. It was observed that lateral shear modulus of subsoil from lateral pile test is approximately 2800kPa to simulate the similar deflection as Table 3.

From the Figure 10, based on simple elastic beam theory with force couple, the end moment for fully restrained pile with one end deflects laterally can be computed as below:

$$\text{Moment} = (PL/2) \tag{4}$$

Where P is imposed lateral load at pile cut-off level and L is the pile length from cut-off level to the river bed, which is 9m for this case study.

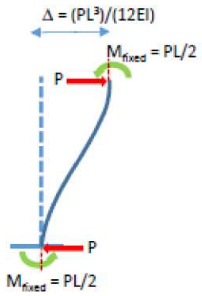

Figure 10 Fully restrained pile with end deflecting laterally

Table 9 Computed lateral deflection (mm) of single lateral pile test and single pile at designed cut-off level

Lateral Force (kN)	**41**	**97**	**138**	**135**
Moment (kN.m)		**Deflection**		
432	4.9mm (L=16.6m)	-	-	-
864	-	10.4mm (L=16.6m)	-	-
1296	-	-	15.3mm (L=16.6m)	-
608	-	-	-	10.2mm (L=9.0m)

Table 10 Computed pile rotation, θ (rad) of single lateral pile test and single pile at designed cut-off level

Lateral Force (kN)	**41**	**97**	**138**	**135**
Moment (kN.m)		**Pile Rotation, θ**		
432	0.002 rad (L=16.6m)	-	-	-
864	-	0.003 rad (L=16.6m)	-	-
1296	-	-	0.005 rad (L=16.6m)	-
608	-	-	-	0.003 rad (L=9.0m)

Based on the back-calculated shear modulus of 2800kPa for a single lateral test pile results, pile group analyses was performed to check on the overall pile group lateral deflection which shall not exceed acceptance criteria of maximum 20mm specified in the contract.

6. CONCLUSION

By performing the interpretation of the static lateral load test results, the pile performance assessment at service condition does yield useful conclusions as following:-

a) The single lateral test pile behaves linear elastic at least until the specified working lateral load. This test pile results are expected to yield conservative lateral pile capacity as much higher extra moment has been inevitably generated to induce premature structural failure of the test pile.

b) For the ultimate limit state of the pile structural capacity, the lateral pile capacity of the test pile is primarily governed by the induced end moment, which is very high in this testing scheme due to overly high free standing length (20.6m) above the riverbed. Such incomparable single lateral pile test setup to verify the actual pile performance at designed pile cut-off level has demonstrated that the pile has sufficient lateral resistance in the structural aspects. Should the free standing pile length be reduced to the actual pile cutoff level, then the pile lateral structural capacity shall be significantly increased and also less lateral pile deflection shall be expected. The structural cracking moment and ultimate moment of the test pile have been verified to be 1,060kN-m and 1,597kN-m respectively.

c) The ultimate lateral pile geotechnical capacity of 560.5kN is well above the specified lateral pile testing load of 135kN.

d) Linear elastic behaviour of the pile structure and geotechnical performance remains valid for the range of lateral loading up to 135kN is also validated. The back-calculated stiffness parameters of the subsoil embedding the test pile are used to assess the working pile lateral deflection for single and then group piles at service condition.

7. REFERENCES

Ken Fleming, Austin Weltman, Mark Randolph and Keith Elson, (2009) “Piling Engineering”.

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