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Design and Construction of Foundation System for Malaysia First Drawbridge at Kuala Terengganu

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ABSTRACT: This paper describes the design and construction of foundation system for Malaysia's first drawbridge, which is located at the river mouth of Kuala Terengganu River. The proposed drawbridge aims to as bridge link connecting the northern and southern areas of the Kuala Terengganu River. With the 76m long main span of bascule bridge decks, it permits no obstruction to the navigation traffic along the river. The design challenges of the drawbridge foundation with free-standing pile length above the river bed are its stringent foundation displacement requirements for the operation of the mechanical bascule structures under the critical loading conditions (i.e. wind load, seismic load, accidental vessel impact and hydraulic load in addition to the structural dead loads, live loads and other imposed operational loads) and also compliances with maritime structure design code of practice throughout the required designed life period. In general, each of the drawbridge tower structure is founded on 118 numbers of 1m diameter with vertical and raked pre-casted marine spun piles. A giant pilecap with dimensions of 76m (long) x 28m (wide) x 4.0m (thick) was designed for load transfer between super-structure and foundation system. During construction works, the validation of the pile performance was very carefully devised and implemented, which included reference compressive static maintained pile load test, static lateral pile load test, Statnamic test, high strain dynamic pile tests on working marine piles over the river. The design and construction processes of ensuring the pile head connection conditions achieving the design rotation stiffness and the problems encountered during massive staged concrete casting of the giant pilecaps with submerged soffit will also be elaborated and discussed in this paper.
Keywords: bridge foundation, drawbridge, marine structure, marine piling

1. INTRODUCTION

This paper shares the experience on design and construction of an alternative marine foundation system learnt from Malaysia's first drawbridge project, which is located at the river mouth of Kuala Terengganu River, with the intended purpose of connecting the northern and southern divided developments across the river. The overall length of the two three spanned integral bridge from both side of river banks approaching the 23 m wide bascule bridge deck on two tower piers is 632 m. With the 76 m long main span of bascule bridge decks on river, it permits no obstruction to the daily navigation traffic along the river. Figure 1 shows the overall layout plan and section view of the project.

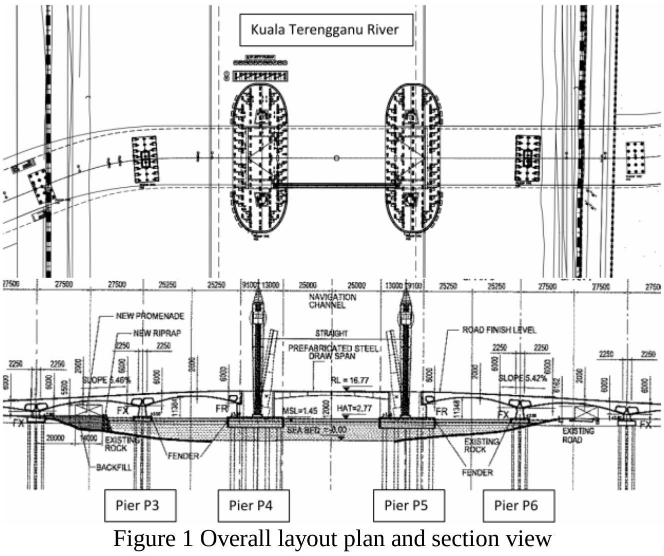

Figure 1 Overall layout plan and section view

The foundation system at marine portion consists of 4 piers, namely P3 to P6. Two heavy foundation at P4 and P5 are designed to support 55 m high drawbridge towers and 76 m long bascule bridge decks and the end span of the integral bridge.

2. HYDROLOGY AND GROUND CONDITIONS

The site is located at Kuala Terengganu river mount and subjects to daily seawater tides from South China Sea. Accordance to the Kuala Terengganu standard port datum, mean sea level (MSL) of the project site was defined at RL0m (+1.45 m Chart Datum (CD)). Meanwhile, the highest and lowest astronomical tide level are at RL+1.32 m (+2.77 m CD) and RL-1.45 m (0.00 m CD) respectively. The riverbed level along the bridge alignment is at about RL-8.3 m (-6.85 m CD).

Based on the Geological Map of Peninsular Malaysia, 8th Edition, published by Minerals and Geoscience of Malaysia in 1985, the site location is generally underlain by Quaternary Alluvium. The site investigation program confirms that the alluvium deposits at this area generally consist of marine deposits with mainly dominant of sand and gravel, but also consists of clay and silt of intermediate plasticity. The STP'N value of subsoil is generally less than 15 for 22 m depth below riverbed and hard residual formation stratum was reported at 21 m to 26 m below riverbed as revealed from the proposed boreholes, in which Figure 2 shows the interpreted subsoil profiles of the riverbed.

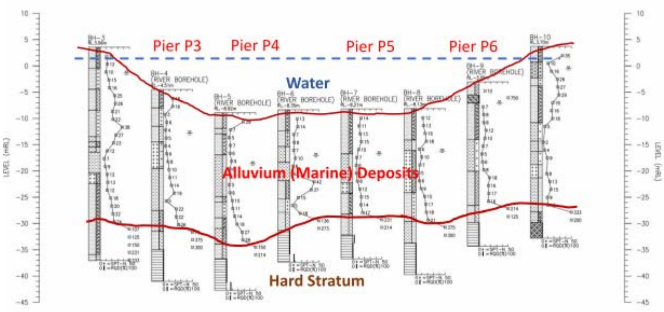

Figure 2 Interpreted subsoil profiles

3. DESIGN CRITERIA

The design criteria of the foundation system mainly focus on operation requirements of drawbridge structure, serviceability conditions and durability of marine structure throughout the designed life period. Interaction design coordination was conducted among bridge design engineer, operation system suppliers and foundation design engineer to establish design criteria for the foundation design. As a result, the foundation system need to design for total settlement less than 10mm and horizontal displacement less than 20 mm in any directions under service conditions.

As the foundations structures are located at river mount near to South China Sea and subject to daily seawater tide, therefore the durability design shall according to the marine structure standard. The foundation structure design including pilecap are in compliance with codes of practice (British standard) BS5400 - Steel, Concrete and Composite Bridge, BS6349 - Maritime Structures and BS8500 –

Concrete. Based on Table 1 in BS5400 part 4, the environment condition of the project is classified as extreme condition. Where the concrete surface exposed to the abrasive action (high and low tidal cycle, and upstream flood storm) of sea water; thus, the design crack width of the reinforced concrete shall limit to 0.1mm in service limit state (SLS) condition. In view of the extreme condition, concrete for the pilecap also to design in accordance with BS8500-1. Exposure class of the pilecap condition are defined as class XS3 in which is subjected to tidal, splash and spray zones as describe in Table A.1 of BS8500-1.

The proposed alternative marine spun pile foundation design was also considered some extreme cases condition such as wind load especially wind load on bascule deck in lift-up condition, seismic load during design earthquake incident, accidental ship impact onto foundation structure and hydraulic forces due to river storm flow.

Other than design considerations mentioned above, construction methodology and sequence were also evaluated to ensure that the proposed method can meet to design requirements at all times. Quality assurance and control measures during construction work are also importance such as temperature control, provision of construction joints for large pour concreting of the gigantic pilecaps.

4. ANALYSIS LOAD CASES

Several load cases in SLS and ultimate limit stage (ULS) were provided by bridge engineer based on their superstructure requirements. Table 1 summaries all the critical load cases in SLS for pile group design and Table 2 summaries all the critical load cases in both SLS and ULS for pilecap design.

Table 1 Critical Load Cases in SLS for Pile Group

No.	Load Case	γ_{fL} considered in load combination						
		DL	SDL	LL	WL	EL	AL	DF
1	SLS-1.0EL	1.0	1.0	1.0	-	1.0	-	-
2	SLS-1.0WL	1.0	1.0	1.0	1.0	-	-	-
3	SLS-1.0AL	1.0	1.0	1.0	-	-	1.0	-
4	SLS-1.0DF	1.0	1.0	1.0	-	-	1.0	1.0

Table 2 Critical Load Cases in SLS and ULS for Pilecap Design

No.	Load Case	γ_{fL} considered in load combination					
		DL	SDL	LL	WL	EL	TL
1	ULS-1.2EL	1.2	1.2	1.2	-	1.2	1.2
2	ULS-1.2WL	1.2	1.2	1.2	1.2	-	1.2
3	ULS-1.4EL	1.0	1.0	1.0	-	1.4	1.0
4	ULS-1.4WL	1.0	1.0	1.0	1.4	-	1.0
5	SLS-1.0EL	1.0	1.0	1.0	-	1.0	1.0
6	SLS-1.0WL	1.0	1.0	1.0	1.0	-	1.0

Where,

γ_{fL} = partial load factor
DL = Dead load
SDL = Superimposed dead load
LL = Live load
WL = Wind load
EL = Earthquake seismic load
AL = Accidental load
DF = Water flow drag force
TL = Traffic load.

5. FOUNDATION DESIGN

The original tender design was cast-insitu bored piles with temporary cofferdam island as working platform for bored piles installation and pilecap construction. An alternative proposal with cost and time effective consideration was proposed using pre-stressed spun pile with marine driven piling method. Upon considering the logistic and constriction feasibility, 1m diameter with grade 80 pre-stressed spun pile (140 mm wall thickness with 7.4 MPa effective pre-stressed) up to 36 m long in single casting length was selected for this project. Pre-welding with extension pile for pile length more than 36 m and pile shoe welding were carried out at pile manufacturing factory and delivered to project site using marine barge. The spun piles are installed by marine piling barge using hammer drive in method.

Based on the manufacturing product information, allowable axial load of 1m diameter marine spun pile is 6850 kN and calculated cracking bending capacity is 835 kNm. The estimation of compression pile allowable geotechnical capacity is generally based on the SPT-N profile using Meyerhof's Method (1984). The ultimate bearing capacity of pile is equal to the sum of the ultimate resistance of the base of the pile and the ultimate skin friction over the embedded shaft length of the pile. The allowable geotechnical capacity of a single pile is generally derived by applying partial safety factors as follows: -

$$F_{p,Geo(allow)} = \frac{f_s A_s}{1.5} + \frac{f_b A_b}{3} \quad (1)$$

$$F_{g,Geo(allow)} = \frac{f_s A_s + f_b A_b}{2} \quad (2)$$

where,

f_b = Base resistance (150 x SPT-N limit to 16,500kPa)
f_s = Shaft resistance (2.5 x SPT-N)
A_s = Surface area of pile shaft
A_b = Surface area of pile base

Estimated pile lengths below riverbed level were based on pile either terminated within hard stratum not beyond bearing materials with SPT-N value more than 200 or resting on bedrock. Calculated allowable geotechnical capacity based on available adjacent borehole results are summarised in Table 3.

Table 3 Calculated Allowable Pile Geotechnical Capacity

Pier No	Reference Borehole	Estimated Pile Length below Riverbed (m)	Allowable Geotechnical Capacity		
			kN	Shaft	Base
P4	BH-5	25	5,851	15%	85%
	BH-6	22	5,966	16%	84%
P5	BH-7	20	5,547	13%	87%
	BH-8	22	5,717	14%	86%

Single pile working load is 5500 kN with major contribution up to 85% from base resistance. Unique pile shoe design was adopted to enhance pile toe structure resistance. Thick steel end plate pile shoe was designed to resist end bearing pressure of pile as per Figure 3.

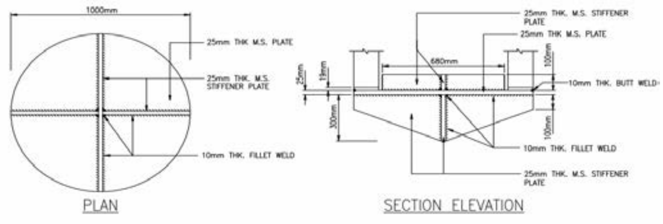

Figure 3 Pile Shoe Details

Pile group analysis was carried out by using soil-structure interaction software, PIGLET developed by M.F. Randolph (2004). Three dimensional pile group deformation as well as forces along the pile can be reasonably well estimated by adopting soil-structure input parameters as summarised in Table 4.

Table 4 PIGLET Input Parameters

Input parameter	Design value
Soil Shear Modulus (Axial), G axial	11.6 MPa
Soil Shear Modulus (Lateral), G lateral	2.8 MPa
Soil Base Modulus, G Base	2.3 GPa
Pile Diameter, D	1 m
Pile Young Modulus, E axial	19.3 GPa
Pile Young Modulus, E lateral	29.3 GPa

Generally, Young Modulus of grade 80 spun pile is taken as 40 GPa and conversion from spun pile to solid circular pile properties need to be calculated as equivalent parameters.

The outcomes of the analysis results indicated that a total 118 nos. of spun pile are required in order to meet the serviceability requirement. Figure 4 shows the piling layout plan of Tower Piers P4 and P5. For enhancement of lateral pile group resistance, 32 nos. raked piles (with inclination 1 Horizontal : 8 Vertical) in the pile group for Tower Piers P4 and P5 were proposed during the pile arrangement configuration. The maximum predicted vertical settlement and lateral displacement in X-Direction (along bridge alignment) and Y-Direction (perpendicular to bridge alignment) are summarized in the Table 6.

Table 6 Summary of Pile Group Deformation Analysis Results

No	Load Case	P4			P5		
		(mm)					
		δ_z	δ_x	δ_y	δ_z	δ_x	δ_y
1	SLS-1.0EL	9.8	1.6	3.1	9.7	1.7	3.3
2	SLS-1.0WL	9.8	17.8	3.9	9.7	19.1	4.3
3	SLS-1.0AL	9.8	7.6	12.9	9.7	8.2	13.7
4	SLS-1.0DF	9.8	7.6	6.7	9.7	8.2	7.3

Where δ_z, δ_x and δ_y refer to pile group displacements (in mm) in vertical settlement, X and Y directions on plan as defined above.

By summarizing the pile group displacement analysis obtained from each load case, it was concluded that the predicted displacements for the final configured pile group arrangement after numerous configuration trials are able to fulfil the foundation displacement requirement where overall vertical settlement are less than 10 mm and lateral displacement in X and Y direction are both less than 20 mm as required.

Fixed pile head connection to pilecap was adopted in foundation system to efficiently control the lateral displacement of foundation system. Fixed pile head connection is defined as a connection details with ability to develop and maintain the restraining end moment at the connection portion under zero rotation condition at the connection interface. For spun pile with hollow annulus section, reinforced concrete pile plug is required to serve as bridging connection element between spun pile and pilecap structure. The tension reinforcements and anchorage length shall be provided to attain the end moment reaction as expected. Typical pile plug details is shown in Figure 5.

6. MARINE PILECAP DESIGN

The proposed pilecap for Tower Piers P4 and P5 are approximate in capsule-like shape with long dimension of 76 m and width of 28 m (see Figure 4). General pilecap thickness is 4m except the centre part reduced to 2 m for accommodating the counterweight block of bascule deck required for operation purpose. Concrete grade adopted is Strength Class C32/40 (cylindrical strength/cube strength) with minimum characteristic cube strength of 40 N/mm^2 at 28 days.

According to BS8500, exposure class of the concrete for marine pilecap shall be classified as class XS3 (tidal, splash and spray zone). To obtain concrete grade C32/40 with 75mm for class XS3, cement type CEM IIB-V (Portland cement with 21-35% fly ash) or cement type CEM IIIA (Portland cement with 36-65% ground granulated blast furnace slag) shall be adopted as supplied concrete. The proposed cement content and water ratio for the concrete mix design (C32/40 with 75 mm for class XS3) are as follow: -

Table 7 Proposed Concrete Mix Design

Subject	Proposed Design	Requirement of BS8500
Ground Granulated Blast Furnace Slag Content	60%	35% - 65%
Cement Content (kg/m3)	415	> 360
Water Ratio	0.4	< 0.45

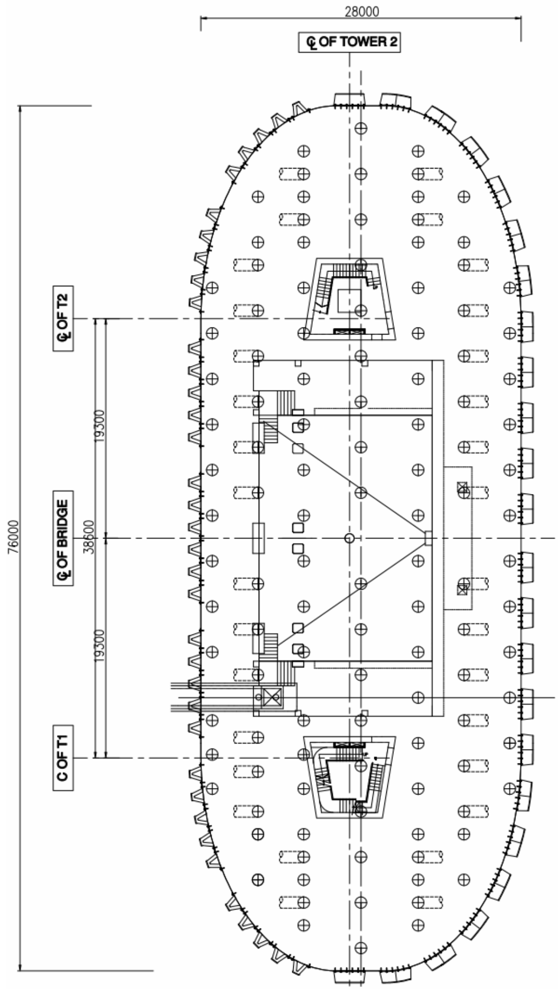

Figure 4 Piling Layout Plan

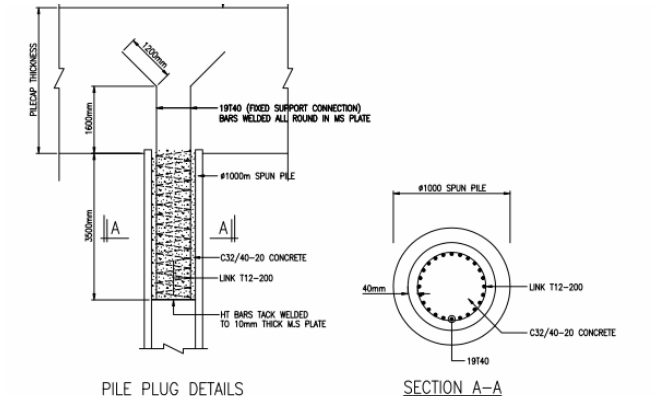

Figure 5 Typical Pile Plug Details

7. DESIGN VERIFICATION TESTS

Design verification tests were conducted to verify the analysis and design assumptions before commencement of production work. The spun piles were designed to take lateral forces and therefore pile bending resistance required to be verified by tests. Pile structural bending test was carried out to obtain allowable pile structural bending capacity as well as pile stiffness derived from pile deflection profile under four-point flexural test. Calculated elastic modulus (E) of Grade 80 concrete is 40 GPa and Flexural Stiffness (EI) is 1,436,000 kNm^2. For the pile concrete durability test, Rapid Chloride Penetration Test (RCPT) in accordance with ASTM 1202-97 was assigned. The result showing the concrete used for spun pile is in compliance with the project specification of charge passing not more than 1000 coulombs, implying very low chloride ion penetrability of the concrete and thus is suitable to be used for the proposed site environment. For environment control, a layer of bituminous coating was required for top 3m from the pilecap soffit level as additional protection.

Continue pile monitoring (CPM) test on pile installation was performed to monitor the impact force and stresses during pile installation until termination to establish the required set criteria. Set criteria was also verified with the performance of mobilised capacity in High Strain Dynamic Pile Test (HSDPT) during end of drive of the test piles. As a result, pile termination criteria adopted for vertical pile is 20 mm per 10 blows using 35 tons hammer with drop height of 0.8 m. Whilst for raked pile is 10 mm per 10 blows after considering hammer drop efficiency in incline position for the necessary pile rake.

An instrumented non-working pile near to Pier P5 was carried out in a maintained load test using reaction pile system with additional deadload of barge over the reaction beam. The main objective of the instrumented test pile is to establish and verify the design assumption of adopted soil parameters. 15 nos. temporary pipe piles were installed at surrounding of test pile as reaction piles. The test pile was expected to load test up to maximum compressive load of 16,500 kN (3 times of pile working load) or failure load whichever comes first. The result of maintained load test is shown in the Figure 6. Pile top settlement at working load is 11.98 mm with residual settlement of 1.94 mm. Pile toe settlement at working load is 2.94 mm which correspondence to soil base modulus of 2.3 GPa (approximate 1150 times of average SPT-N value for PIGLET analysis). Back analysis from the MLT results for soil shear modulus in axial is 11.6 MPa (approximate 1450 times of average SPT-N value for PIGLET analysis).

The foundation for all over water Piers, P3, P4, P5 and P6 are subjected to lateral load. Lateral shear modulus, $G_{lateral}$ adopted for lateral deformation and also the later pile capacity in the foundation design shall also be subjected to field test verification. Lateral pile load test (LPT) had been performed to assess the pile performance in both structural and geotechnical capacity as well as soil stiffness and pile deformation under lateral action.

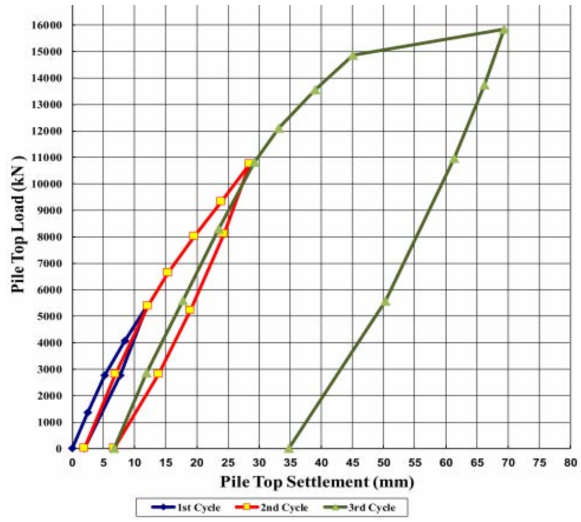

Figure 6 Maintained Load Test Result

The LPT is able to verify the design assumptions on prediction of lateral pile group movement under service loading condition (Serviceability Limit State Check). The tested pile length is 38 m long instrumented with inclinometer at centre of spun pile annulus for lateral displacement monitoring. In addition, linear variable differential transformer sensors were attached on a separate reference column to measure the lateral pile head deflection. 5 nos. temporary pipe piles were installed in line with test pile as reaction piles. Figure 7 showing setting out arrangement and the LPT results are presented in Figure 8. In the first loading cycle, the observed maximum pile head deflection at the tested load of 137.8 kN was 147.53 mm. Upon unloading to zero, the pile rebounded to a residual deflection of 18.32 mm. In the second loading cycle, the observed maximum pile deflection at the tested load of 220.0 kN was 398.67 mm for the test pile.

Figure 7 Lateral Pile Test Result

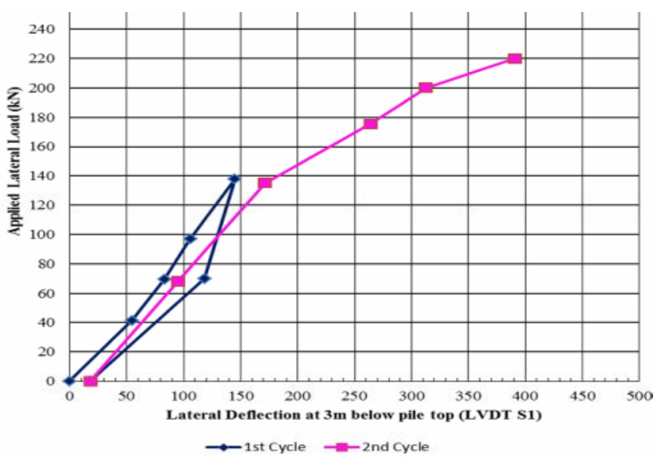

Figure 8 Lateral Pile Test Result

Back-analysis was carried out with computer software, PIGLET to compare the assumed subsoil lateral shear modulus with the actual pile lateral deformation. It was observed that actual lateral shear modulus, $G_{lateral}$, of subsoil from lateral pile test is approximately 2.8 MPa (approximate 350 times of average SPT-N value for PIGLET analysis), which is relatively lower than the assumed lateral shear modulus of 4.3 MPa in the design stage.

With the two preliminary pile tests under compressive loading and lateral loading carried out, design assumptions and adopted subsoil parameters can be verified and calibrated for foundation design. With the obtained design parameters and review of the four river pier pile group performance, revised detailed design with more piles was needed for Pier P3 to ensure compliance pile group performance under the assigned loading.

During the working pile installation, 2 nos. of statnamic tests were carried out for Tower Piers P4 and P5 as part of the quality assurance/ quality control (QA/QC) procedure. Outcomes of the statnamic load test results showed mobilised test load with force pulse duration satisfied the condition stated in ASTM D7383-10: (1) The applied force shall exceed the pre-load for a duration time of at least twelve times the test pile length (L) divided by the strain wave speed (c), 12L/c, and (2) The applied force shall exceed 50% of the actual peak force for a minimum duration of four times L/c.

HSDPT had also been carried out as QA/QC procedure for installed working pile. Total of 22 Nos. (9.3% of installed piles at P4 & P5) HSDPT were conducted with mobilised test load of up to 2 times working load.

Meanwhile, verification test for designed concrete is vital before the concrete casting work on site in order to prevent concrete quality issue of the structure along the structure's service life. Series of test such as concrete slump retention test, concrete casting temperature monitoring, concrete trial mix and concrete cube strength test were carried out prior to pilecap execution works on site.

Large volume concrete casting (casting thickness more than 1m) was anticipated at the initial stage of project and the temperature development during casting was always the major concern for the pilecap construction work, thus, temperature monitoring was proposed during initial stage concrete casting at Pier P3. Designated initial concrete temperature was checked with estimation of 12˚C heat generated from every 100 kg/m^3 Portland cement hydration process and thus determining 27˚C as the concrete initial temperature; to control maximum concrete temperature below 75˚C during curing. However, the designed temperature is then further verified with on-site concrete temperature monitoring.

Post construction instrumentation and monitoring was proposed upon completion of the pilecap casting works. Four (4) settlement markers were installed on each pilecap to monitor the foundation performance during construction stage of the superstructure. The instrumentation monitoring results show maximum cumulative foundation settlement of 5 mm with substantial dead load existed on the bridge in January 2018. The instrumentation monitoring period will be extended up to completion of superstructure construction works.

8. CONSTRUCTION

The proposed Piers P4 and P5 are located in Kuala Terengganu River and the piling works for the foundation system was installed by adopting marine piling method i.e. hammer driven from piling barge. Several constructions aspects needed to be taken into consideration for the marine piling works such as correct positioning and stability of piling barge during pile handling and installation process under the impacts from active river flow, daily tidal changes from South China Sea, unpredictable strong wind gust in the distinct monsoon season at east coast of Peninsular Malaysia. Aforementioned conditions have created work challenges and safety issue of work, thus posing uncertainty in overall construction progress. Addition precaution measures and construction planning are considered to prevent any unwanted incident and delay during construction works.

Pre-joined one-length spun pile was decided with construction and design consideration. From construction aspect, one-length pile is commonly use in marine piling with time saving for handling and avoidance of inconsistency of site welding of pile joint. The delivered pile lengths were in 32 m and 38 m as estimated from adjacent boreholes information plus extra 3m for construction tolerance.

Some pile performance verification tests were conducted in river. Temporary work preparation for the pile tests were properly planed with a safe temporary platform for the sole purpose of testing. In general, working platform level is required to be above Highest Astronomical Tide (HAT) level to be free from the tidal and wave impacts.

The pile foundations are designed with fixed head condition between pile head and pilecap. Load transfer element with introducing 3.5 m pile plug inside the spun pile anulus of sufficient anchorage reinforcements into pilecap. Effectiveness of pile plug performance is much dependent on installation workmanship. A layer of weak cement laitance with primary cement/lime slurry will normally found around the pile inner anulus due to segregation of excessive cementitious slurry and aggregate compaction from the spinning process. Therefore, the contractor shall ensure proper removal of this weak cement laitance before casting of pile plug.

During piles installation, unexpected weak soil stratum was encountered at Pier P4 causing longer pile penetration length than estimated pile length. As such additional extension piles with on-site pile joint welding was needed. In view of potential insufficient bending capacity and flexural stiffness at pile joint area, longer reinforced pile plug was proposed to enhance the pile joint capacity.

After completion of piles installation works, an unforeseen incident case was encountered at Pier P5. A pile head was observed missing during site inspection. As the regular river traffic for public has to be uninterrupted between P4 and P5, it was suspected the pile was damaged by uninformed accidental ship impact occurred during the period of off-construction works. Underwater inspection had been carried out and the missing pile head was found broken and estimated the damage at 9.5m below pile cut off level (2.5 m below riverbed level). Remedial works had been explored with permanent steel casing extended up above water level. The material falls into pile annulus required to be cleaned up for placement of reinforcement cages in the remedial works by tremie concreting in the cleaned pile annulus from pile toe to the pile cut off level.

The proposed pile cap soffit level of Piers P4 and P5 is RL-0.5 m, which is 0.5 m below the MSL. Underwater concrete casting for the pile caps was anticipated. Several options had been explored and proposed by contractor for underwater concrete casting method. The final adopted temporary work for pile cap concrete casting is watertight steel formwork mounted to the group piles with water seals around the piles. Before placing of steel formwork, vertical support at the piles is required to carry the vertical loading from temporary formwork and self-weight of the lean concrete mentioned later. The designed supports consist of prefabricated steel gripper on each installed pile head and some secondary girders supporting the steel form. In order to provide sufficient bonding between the piles and steel grippers, a layer of synthetic rubber was used. Bonding test was carried at laboratory for design certification. The steel gripper was locked by competent diver under water with bolt and nut tightening system. After that, steel formwork will progressively place over the support system with a layer of 150mm thick lean concrete below pile cap soffit level. Dewatering from inner side of steel formwork was then carried out to create dry working condition for pile cap concreting work. For safety consideration, the water pressure outside the steel formworks was considered at HAT level. Figure 9 shows the pile cap concrete casting preparation works.

The pile cap thickness of the pile cap Piers P4 and P5 is 4 m thick with total concrete volume of 7,025 m^3 for each pile cap. With consideration of local concrete supplier availability and the friction limitation of the steel gripper support, concrete casting had been divided into 4 stages. Stage 1 – cast a layer 700 mm thick concrete throughout entire pile cap footprint (concrete volume of 1,353 m^3) to serve as base support for subsequence concreting work. Stage 2 –

cast a layer of 1,300 mm thick concrete (concrete volume of 2,124 m^3) on top of completed base layer. Stage 3 – cast a layer of 1,000 mm thick concrete (concrete volume of 1,774 m^3) except centre portion, in which a void was deliberately formed to house the counterweight block of the bascule deck. Stage 4 – final cast with layer of remaining 1,000 mm thick concrete (concrete volume of 1,774 m^3) up to finished pile cap top level. By adopting stages concrete casting method, cold joint between earlier cast concrete and new cast concrete was expected. With that, the flexural horizontal shear stress against the interface shear resistance capacity shall consider in pile cap design, and thus horizontal shear resistance was enhanced by placing shear key reinforcements across the entire cold joint surface.

Pump concrete method was adopted for marine pile cap casting. Pumped mixed concrete at batching plant was delivered to riverbank via concrete truck. After that, the concrete was pumped to pile cap through a 100 m long steel pipe with temporary support over the river. Two concrete pumps had been deployed on site where one pump was on standby in case any pump breakdown to maintain uninterrupted concrete casting. Workability and pumpability of concrete also been considered for large volume and long duration concrete casting. The supplied concrete workability had been specified to have minimum 4 hours slump retention period with consideration of maximum 2 hours concrete delivery time and maximum 2 hours concrete pumping and placing time.

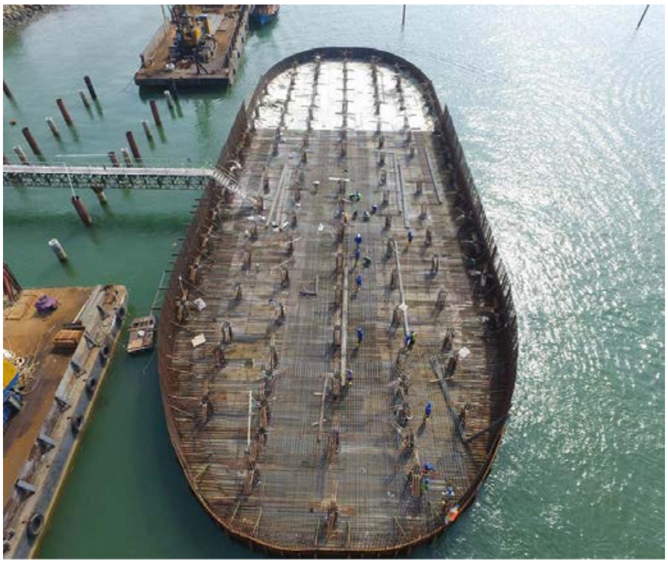

Figure 9 Pile cap concrete casting preparation works

For concrete temperature control, heat generated during hydration process shall be controlled not exceeded 70°C and change in gradient not exceeded 20°C. Based on the temperature monitoring result at P3 pile cap shown in Figure 10, heat generated for 2,350 mm thick pile cap is 40°C with 415 kg/m^3 cement content. Therefore, the initial temperature during concrete mixing shall be controlled below 30°C. From the concrete mix design, free water requirement is designed to be 165 kg/m^3 with about 40% of water will be substituted by ice in order to lower down the initial temperature of concrete. For quality assurance and quality control, initial temperature of concrete and slump had been measured and recorded. Concrete temperature or slump test exceeding specified requirement was not allowed to proceed for concreting.

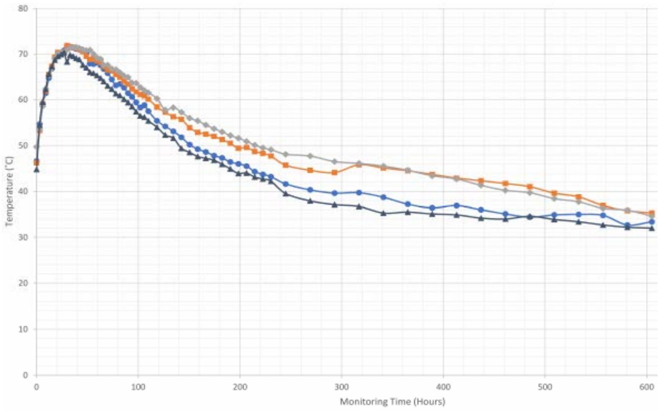

Figure 10 Concrete temperature monitoring

9. CONCLUSION

Alternative foundation design work was started since year 2014 when project was awarded. Regular meetings and coordination workshops were conducted between bridge engineer and foundation engineer to establish an appropriate alternative foundation system as well as foundation contractor for ease of construction for Malaysia first drawbridge project. Several loading cases were considered in foundation design to meet the serviceability and operation requirements of bridge superstructure. Environmental and marine conditions and potential worst design condition of the project were also taken as part of design and construction evaluation. Cost and time effective foundation system with driven pre-stressed spun piles was developed purposely for this project to save both time and cost of the project. Physical foundation construction work was commenced in early of year 2015 and completed in end of year 2016. Superstructure work including the integral bridge, bascule deck and tower structure are in progress in year 2018. Figure 11 shows the latest overall site condition in Feb 2018.

Figure 11 Progress in Feb 2018

10. REFERENCES

British Standard Institute (BSI). (2006) BS8500: Concrete – Complementary British Standard to BS EN 206-4.

British Standard Institute (BSI). (1988) BS5400: Steel, Concrete and Composite Bridges.

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

Liew S.S., Koo K.S., Chee F.W. and Tan P.Y, (2016) "Planning and Interpretation of Instrumented Lateral Pile Test Performance with a Semi Restrained Pile Head Condition"

M.F. Randolph, (2004) "PIGLET Analysis and Design of Pile Groups".