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Aspects of the Foundation Design for the Merdeka 118 Tower, Malaysia

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ABSTRACT: The Merdeka 118 Tower in Kuala Lumpur Malaysia is currently the second-tallest building in the world. This paper describes the process involved in the geotechnical design of the foundation system for this super-tall building. The results of the geotechnical investigations are summarized and the development of a geotechnical model is described. The following aspects of the preliminary foundation design are then discussed: the overall stability of the tower foundation system, the geotechnical capacity of the piles, the total and differential settlements, the assessment of foundation stiffness values for structural design, the dynamic stiffness and damping of the foundation system, the seismic assessment of the site. Four preliminary pile tests were carried out on instrumented piles using bi-directional load cells (Osterberg Cells). The test results revealed that the performance of the tested piles was less favourable than had been anticipated in the previous design phases, and so the pile design parameters were modified accordingly in order to achieve the same capacity as estimated in the schematic and pre-final design phases. The final design comprised 136 reinforced concrete bored piles, with diameters of 2.2 m and 2.4 m, and corresponding lengths of 60 m and 52 m. The estimated long-term settlements were 120 to 130 mm for the structural scheme adopted. It was anticipated that most of the settlements would occur during the construction of the tower.

KEYWORDS: Analysis, dynamic response, foundation design, piled raft, settlement.

1 INTRODUCTION

This paper discusses various aspects of the geotechnical foundation design for the Merdeka 118 Tower in Kuala Lumpur, Malaysia. This tower is now the second-tallest completed building in the world, and is located near the Stadium Merdeka in Kuala Lumpur. The tower was initially designated as KL100, and was proposed to consist of in excess of 100 storeys with six to ten levels of basement, but the tower was later extended to 118 storeys. The building footprint is approximately 3340m^2 in area and is diamond-shaped in plan. A piled raft foundation was considered to be appropriate for this tower, since the ground conditions were assessed to be favourable for this foundation type.

The paper sets out the design process followed initially, and the results obtained prior to a load test program being carried out. After the load test results were interpreted and assessed, it was found that the performance of the test piles was less satisfactory than had been assumed for the design. Accordingly, the foundation design was modified and the ensuing design was termed the “Final Design”, while the previous design was termed the “Pre-Final Design”.

2 GEOTECHNICAL INVESTIGATIONS

Based on available geological and geotechnical information, the tower site is underlain by the Kenny Hill formation, overlying the Kuala Lumpur Limestone formation. The Kenny Hill Formation is thought to be Permian to Carboniferous in age and consists of a series of sedimentary rock strata such as mudstone, shale, phyllite, sandstone, siltstone, quartzite, and chert. The rocks of the Kenny Hill Formation are generally extremely weathered and weak, resembling hard soil conditions.

The Kuala Lumpur Limestone appears to dip steeply below the Kenny Hill Formation in the vicinity of the geological boundary, based on case studies examined in past projects.

A series of geotechnical investigations were carried out in 2010 and 2011, and involved the following components:

- The drilling of 9 boreholes and SPT testing, 5 of which extended to 110 m.
- Falling head tests in 2 boreholes.
- Pressuremeter testing at 5 m intervals in 3 of the deep boreholes.
- Downhole vertical seismic shear wave profiling in the 5 deep borehole.
- A grid of multi-channel analysis of surface wave (MASW) lines across the core area.

Table 1 shows the schematic stratigraphic model developed for the site on the basis of these investigations..

Table 1: Schematic stratigraphic model

Unit	Top of Unit RL (m)	Thickness (m)	Description
1	+56.5	0.5	Sandy & clayey gravel
2/3a	+56.0	16.0	Firm to v. stiff clayey silt, gravelly silt, silty gravel. silty clay (SPT <50)
3b	+40.0	35.0	Decomposed rock; v. stiff to hard clayey silt, gravelly silt, silty gravel & sand (SPT>50)
3c	+5.0	40.0	Rock; HW-MW metasedimentary rock with some XW zones. Intact strength very low-medium.
3d	-35.0	>15.0	MW- SW metasedimentary Rock; interbedded metasandstone & siltstone, with some phyllite bands. Intact strength medium-high.

A suite of laboratory testing was carried out and included soil index tests, direct shear and triaxial strength tests, soil chemical content tests and rock strength tests (point load and unconfined compressive strength).

The triaxial strength test data from 19 tests on the residual soils are summarized in Table 2.

Table 2: Summary of triaxial strength tests

Parameter	Unite 2/3a	Unit 3b
Cohesion c’(kPa)	2-5	5
Friction angle ϕ’ (degrees)	26	30

The unconfined compressive strength (UCS) of the rocks were assessed using results of the Uniaxial Compressive UC) test and point load index strength (PL)tests. A calibration factor of 24 was used to convert the latter values to UCS tests. Table 3 summarizes the results of unconfined compression tests carried out. Greater reliance was placed on the UC test results.

Table 3: Summary of rock strength results

Unit		Maximum UCS (MPa)	Minimum UCS (MPa)	Average UCS (MPa)
3c	UC test	22.6	10.2	17.9
	PL test	20.4	12.7	15.7
3d	UC test	24.7	11.0	19.2
	PL test	20.2	14.9	16.5

Figure 1 shows measured values of shear wave velocity V_s versus depth from 5 boreholes. V_s ranges from about 350 m/s to 800 m/s, with a tendency for V_s to increase with depth. Corresponding values of P-wave velocity range between about 1500 and 2750 m/s.

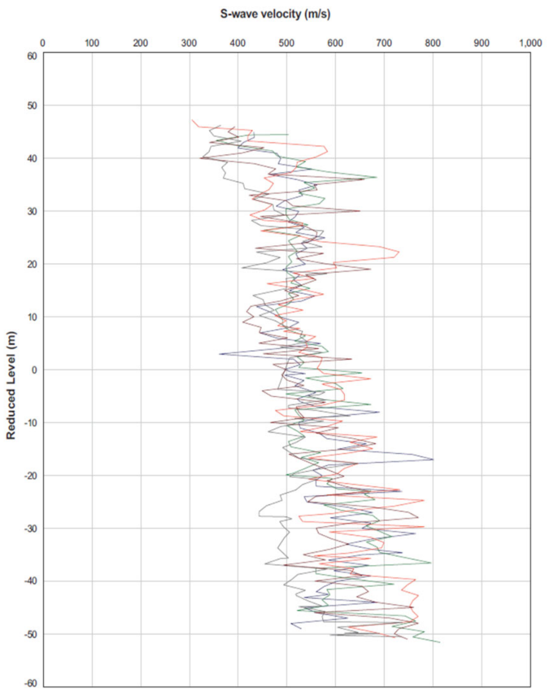

Figure 1: Shear wave velocity versus depth

3 GEOTECHNICAL MODEL AND INITIAL PARAMETERS

The geotechnical design parameters initially adopted were developed from the available geotechnical in-situ and laboratory data using various published empirical correlations, and are shown in Tables 4 and 5. As discussed later in the paper, the results of pile load tests indicated that these initial parameters may have been over-optimistic, and so the final design parameters were modified accordingly.

Based on groundwater measurements on standpipe piezometers, the estimated groundwater level within the site foot[print varied from about 9 m to 13 m below the ground level.

Table 4: Summary of initial pile design parameters for schematic design

Unit	Ult. Skin Friction (kPa)	Ult. End Brg. (MPa)	Lim. Lat. Pressure (MPa)	Young's Modulus (MPa)
2/3a	60	-	-	150
3b	150	3 (raft)	3	250
3c	250 (above RL-5m)	-	3	350
	300 (below RL-5m)	8	4	
3d	500	20	10	450 (above RL-45m)
	"	"	"	1500 (below RL-45m)

Table 5: Summary of strength and permeability parameters

Unit	Effective Cohesion (kPa)	Effective Friction Angle (deg)	Permeability (10^{-6} m/s)
2/3a	2-5	26	3-30
3b	5	30	2-6

4 PRELIMINARY FOUNDATION DESIGN

4.1 Foundation Layout

The foundation system developed for the preliminary design stage involved the use of 136 bored piles, 2.4 m in diameter with a length of 40 m, and 2 m diameter with a length of 55 m. The pile head level was at RL +28.5 m, i.e. about 28 m below original ground level. The piles were supported by a 4 m thick raft, diamond in shape. The larger diameter piles were located near the outer parts of the foundation system, where the axial loads due to wind loading were relatively high. Figure 2 shows a plan of the preliminary foundation system.

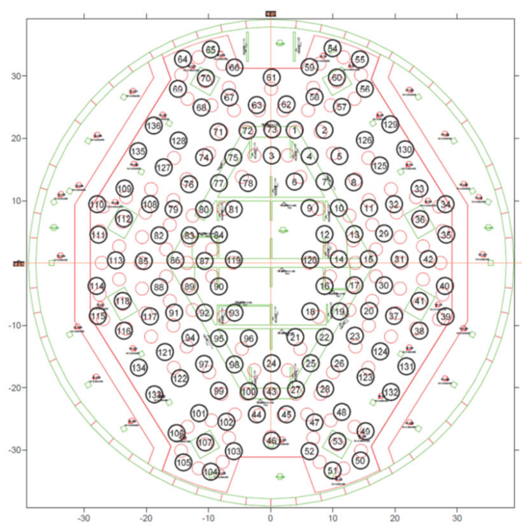

Figure 2: Pile layout for preliminary design

4.2 Overall Stability

An assessment of the overall stability was undertaken using the in-house program "CLAP" (Combined Loading Analysis of Piles). This program computes the distributions of axial and lateral deflections, rotations and loads at the head of a group of piles subjected to vertical loads, lateral loads, moments and torsion. The parameters shown in Table 2 were used to analyze the group shown in Figure 1, with a geotechnical reduction factor of 0.6 applied to the skin friction, end bearing and ultimate lateral resistance values.

Overall stability was satisfied if the foundation system did not collapse when subjected to a prescribed series of ultimate load combinations. These load combinations were supplied by the structural designer, and included the following:

- DL + LL
- 1.2 x DL + F_1xLL(rd) ± 1.6xWL
- 0.9 x DL ± 1.6xWL + H
- 1.2xDL + F_1xLL(rd) ± E
- 0.9 x DL ± E + H

where DL = Dead Load, LL = Live Load, WL = Wind Load, H = Hydrostatic load, E = Earthquake load, and F_1 = live load reduction factor.

The analyses indicated that the overall stability condition was satisfied for all load combinations considered.

4.2 Cyclic Loading

Wind loads for the Tower structure were quite severe, and therefore in order to assess the effect of cyclic loading, an assessment was undertaken based on the approach suggested by Poulos and Davids (2005), in which adequate foundation performance under cyclic loading is achieved provided the following criterion is met:

$$\eta R_{gs}^* \geq S_c^* \quad (1)$$

where R_{gs}^* = design geotechnical shaft capacity, Sc* = half amplitude of cyclic axial wind-induced load, and η = a factor taken to be 0.5 here.

To assess the half amplitude of cyclic axial wind induced load, the difference in pile load was calculated between the two critical wind load cases provided by the structural designer. The results indicated that the criterion set out above was satisfied, and that degradation of shaft capacity due to cyclic loading was unlikely to occur.

4.3 Settlement Estimation

The settlement of the pile group under combined dead plus live loading was estimated using three approaches:

- The in-house program GARP (Geotechnical Analysis of Raft with Piles).
- The equivalent raft approximation.
- The equivalent pier approximation.

From GARP, the estimated maximum and minimum settlements were 88 mm and 21 mm. The equivalent raft approach gave an estimated average settlement of 104 mm, while the corresponding value from the equivalent pier approach was 99 mm. Given the considerable simplifications involved in the equivalent raft and pier methods, the level of agreement was considered to be reasonable.

4.4 Dynamic Stiffness and Damping

Analyses were undertaken to estimate the vertical and lateral stiffness and damping of the tower foundation system, and two approaches were adopted:

1. The in-house computer program CLAP was used to compute the overall static stiffness values for the entire foundation system. For this analysis, typical values of vertical load, lateral load and moment have been applied to the group. This analysis provides overall group stiffness values for static and low-frequency loadings.

2. A dynamic pile group analysis was undertaken in which the approximate approach outlined by Gazetas (1991) was employed. This approach involved the use of dynamic pile stiffness values for individual piles and dynamic interaction factors. This analysis could provide overall group stiffness and damping values, and their variation with the frequency of the applied dynamic loading. To make allowance for the fact that the ground stiffness is greater for dynamic loadings than for static loadings, the values of Young's modulus of the various layers were increased by a factor of 2.

It was found that, because of the simplified soil profiles for which Gazetas' expressions are derived (an infinitely deep soil layer), the values of stiffness in particular may not reflect accurately the overall foundation behaviour in more realistic profiles in which the stiffness increases with depth. Furthermore, the dynamic pile group analyses indicated that the foundation stiffness was little affected by dynamic effects for the range of loading frequencies likely to be of interest in this case (up to about 0.5 Hz). Accordingly, for the dynamic group stiffness, the values from the CLAP analysis were adopted, assuming they are frequency–independent.

Foundation damping arises from two sources: radiation damping, via the dissipation of energy into the soil and away from the foundation, and internal or hysteretic damping of the ground itself, due to its non-linear stress-strain response. The solutions of Gazetas (1991) were used to estimate the radiation damping, despite the considerable simplification involved in representing the soil profile as a homogeneous layer. Gazetas' solutions indicate that radiation damping is likely to be very low if the range of frequencies of loading is less than the natural frequency of the soil profile. For the ground model adopted, the computed natural frequency was about 1.7Hz, and thus was considerably in excess of the likely range of loading frequencies arising from wind loading. Accordingly, any damping that may occur will arise largely from internal damping.

There were no laboratory or field data available that provide direct measures of internal damping for the Kenny Hill formation at this site, nor did there appear to be any available published data. It was therefore assumed that the internal damping ratio of the soil was 0.02, which was felt to be reasonable for a stiff ground profile. It was found that the damping ratios for translational motions were significantly larger than those for rotational motions, a fact that is now well-recognised.

4.5 Seismic Design Issues

It was recognised that, for super-tall structures such as the Merdeka 118 tower, wind loading generally provides the critical lateral loading condition rather than earthquakes. Nevertheless, it was necessary to assess the seismic hazard for the site and to provide information that may be used to estimate earthquake-induced forces and moments on the structure and the foundation. The following process was adopted:

1. Based on the work of Petersen et al (2004) and Adnan et al (2006), estimated peak ground accelerations are as follows: 500 year return period: 0.074g; 2500 year return period: 0.149g.
2. Site response analyses were undertaken to assess the potential for amplification of bedrock motions through the ground profile, and also to estimate the maximum lateral ground movements during an earthquake event. These

movements were then used to assess the kinematically-induced bending moments in the foundation piles.

3. Use was made of the PEER database of earthquake records available online, and an attempt was made to identify records that were from relatively distant sources and had a similar bedrock response spectra to those derived from Adnan et al (2006).
4. 7 records were selected , with the peak acceleration being in the range 0.100g to 0.185g, which was broadly consistent with the range suggested by Petersen et al (2005).
5. The computed response spectra showed that there is an amplification of the peak spectral acceleration for relatively low structural periods. For tall buildings however, where the natural period is large (and in this case of the order of 8s or so), the amplification of motion is very small. The computed amplification factors for spectral acceleration were as large as 3.2, but those at a period of 8s were between 1.01 and 1.09.
6. The pseudo-static approach developed by Tabesh and Poulos (2001) was used to estimate the kinematically-induced bending moments in the piles, using the computed maximum ground movements from the site response analyses in (2) above. The maximum moment occurred at the pile head (which was assumed to be fixed against rotation because it is rigidly attached to the raft), and was governed largely by the imposed inertial force, with the largest computed value being about 3.4 MNm, which was well within the capacity of a 2.1m diameter pile. The bending moments lower along the pile were dependent primarily on the earthquake-induced ground movements, and were relatively small, indicating that, in this case, kinematic effects due to these ground movements were relatively unimportant.

5 PILE TESTING

A comprehensive vertical, lateral and cyclic pile load testing programme was proposed for the KL100 Tower project. Pile load testing is invaluable in confirming design assumptions and finessing the foundation design, and from the pile instrumentation, detailed information can be derived for the distributions of shaft friction and soil stiffness at various depths along the pile shaft. The objectives of the proposed pile load tests were as follows:

• To assess the constructability and integrity of the piles using the proposed construction techniques;

• To allow comparison of measured pile performance with design expectations and refinement of the geotechnical parameters adopted in design (e.g. ultimate skin friction and end bearing values).

Two instrumented test piles (TP3a and TP4) using bi-directional load cells (Osterberg Cells) were constructed within the Tower site. Four vertical pile tests were to be carried out, and the piles were tested to the estimated ultimate load. In addition to the load tests on the Tower piles, an additional test, TP1, was carried out on a podium pile. The results of this test are also considered in assessing the pile performance. The test piles were constructed under bentonite.

Figure 3 shows the interpreted ultimate shaft friction values from the load tests on TP3a and TP4, together with the design values selected for the schematic design. The results from TP3a are extremely low along the upper part of the shaft and are indicative of problems with the construction of the pile. The low values may have reflected problems with the construction process. For TP4, modifications were made to the specified construction process in order to try and achieve larger values of shaft friction. These modifications, which included roughening of the ground around the shaft, appeared to have been beneficial, as the results from TP4 were considerably larger down to about RL-20m, although thereafter, they were similar to the values from TP3a. Unfortunately, the measured shaft friction values in the Unit 3c material were lower than those adopted for the schematic design, and accordingly it was deemed necessary to reduce the shaft friction values, f_s. The recommended design values are shown in Table 4.

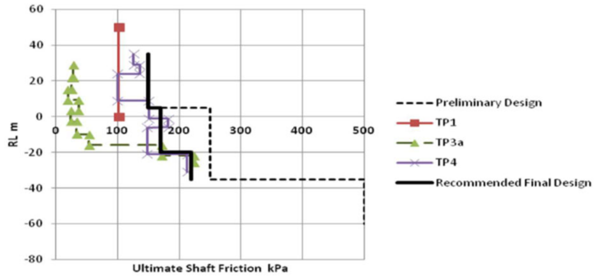

Figure 3: Values of ultimate shaft friction

The maximum mobilized end bearing pressure, f_b, was found to be as follows:

- TP1: f_b = 9.2 MPa (not fully mobilized, base at about RL 0);
- TP3a: f_b = 3 MPa (but far from full mobilization);
- TP4: f_b = 13.8 MPa (not fully mobilized).

On the basis of the above data, an ultimate end bearing value of 12 MPa was recommended for final design.

Figure 4 shows the values of Young's modulus (Es) derived from the pile load test data, together with the values selected for the pre-final design. Down to about RL0, there is a general similarity between the design values and those from the pile tests, but below that level, there is a tendency for the measured values to be less than those used for the pre-final design. Accordingly, the Young's modulus values were amended as summarized in Table 6.

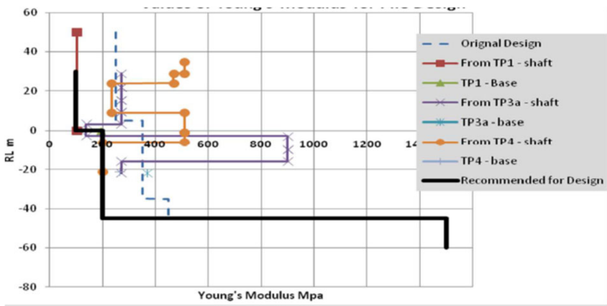

Figure 4: Values of Young's modulus for design

Table 6: Summary of revised design parameters

RL Range (m)	Ultimate shaft friction (kPa)	Ultimate end bearing (MPa)	Young's modulus (MPa)
+28.5 to +5	150	-	100
+5 to -20	170	-	200
-20 to -45	220	-	200
Below -45	-	12	1500

Figure 5 shows the computed load-settlement curve for TP4 when the above Young's modulus values are used. This curve is in better agreement with the load-settlement curve derived from then load test, and bearing in mind that the latter involves a number of approximations for its derivation, it seem reasonable to adopt the above-recommended values of Young's modulus for final design.

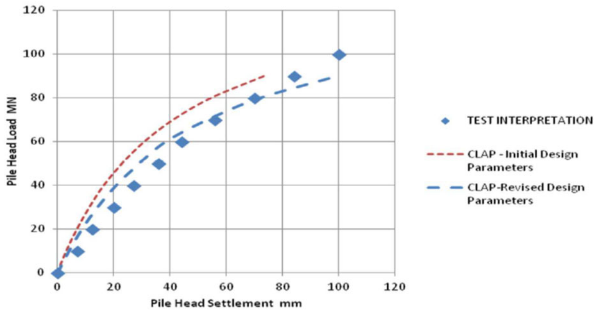

Figure 5: Comparison between load-settlement curves

7. FINAL FOUNDATION DESIGN

The pile load test program revealed that the performance of the piles tested was less favourable that was anticipated in the preliminary design phase. Accordingly, the foundation design was modified. The same foundation layout as that shown in Figure 1 was retained, but the pile lengths were increased such that the 2.4 m diameter pile length increased from 40 m to 52 m, and the 2.2 m diameter pile length increased from 55 m to 60 m.

7.1 Settlement Estimates

The program GARP (Small and Poulos, 2007) was used to estimate the long-term settlement of the foundation system under the combined dead plus live loads. The estimated maximum settlement was 119 mm and the estimated maximum differential settlement was 59 mm. The largest computed angular rotation below the tower footprint was in the order of 1 in 400. However, it was considered that much of the settlement would occur during construction.

7.2 Finite Element Analyses

A 3-dimensional finite element analysis using the commercial software PLAXIS 3D was performed to assess the overall performance of the foundation. The raft was modelled as a linear elastic material and the piles were modelled by beam elements as embedded piles. The soil layers were represented by a Mohr-Coulomb model and the cofferdam was modelled via structural wall elements. Analysis was carried out for the serviceability loads – combined dead plus live loads. PLAXIS takes into account the construction sequences in the analysis and the following construction stages were modelled in the analysis:

• Stage 1: Installation of the cofferdam wall along the perimeter of excavation;
• Stage 2: Installation of the foundation piles;
• Stage 3: Excavation to the base of the raft;
• Stage 4: Construction of the raft;
• Stage 5: Application of the structural loads – dead plus live loads.

Table 7 summarises the estimates of the key foundation performance by PLAXIS 3D and compares these with the corresponding values obtained from the GARP analyses. The computed settlements were similar, but the computed axial pile loads were more extreme from the PLAXIS3D analysis.

The computed lateral movements under wind loading were small, ranging from 5 to 7 mm, depending on wind direction.

Table 7: Estimates of foundation performance

Quantity	*Maximum Value*		*Minimum Value*	
	GARP	*PLAXIS3D*	*GARP*	*PLAXIS3D*
Settlement (mm)	119	128	60	70
Differential settlement (mm)	59	58	-	-
Axial pile load (MN)	42	60	16	14

8 EFFECTS OF FUTURE DEVELOPMENTS ADJACENT TO SITE

Adjacent to the Merdeka 118 Tower site, there were plans for the Merdeka Station MRT development, which would include the following main construction activities:

1. Excavation for the platform cavern, which would be about 35 m away from the tower footprint.
2. Some excavation adjacent to the tower to provide access to the MRT.

The impact of each development (i.e. KL Tower and MRT) on the other would depend heavily on the relative timing of their construction. Tunnel construction and excavation for the platform cavern were expected to occur more or less concurrently. The excavation for the MRT development would have the potential to cause some vertical and lateral ground movements and hence induce additional bending moments in the piles, and could also cause ground movement which could affect the basement and basement support system. However, considering the relative distance of each development from the tower, the preliminary assessment was that the impact on the tower foundation performance and integrity would be small.

9 CONCLUSIONS

The preliminary foundation design was based on the existing available geotechnical information, and it was assessed that 136 piles, 2.1m in diameter and approximately 55m in length with a founding level of RL -35m would be required to support the structure.

Four pile load tests comprised tests on instrumented piles using bi-directional load cells (Osterberg Cells), and the results were incorporated into the final design. The tests revealed that the performance of the piles tested was less favourable than was anticipated in the previous design phases. Accordingly, the pile design parameters were modified in the final design phase and the pile layout was revised accordingly in order to achieve the same capacity as estimated in the preliminary design stages.

The final design pile layout comprises 136 reinforced concrete bored piles, 2.4m diameter with 52m length (outer piles) and 2.2m diameter with 60m length (inner piles). The thickness of the raft was 4m.

Assessments of the ultimate limit state stability under static and cyclic loadings showed the performance of the foundation system to be satisfactory. The predicted long-term settlement was in the order of 120 to 130mm for the structural scheme adopted, and it was anticipated that the majority amount of settlement would occur during the construction of the tower.

This project clearly demonstrated the importance of undertaking pile load testing to verify the foundation design assumptions. In this case, the testing indicated that the design assumptions were optimistic and that a more robust final design was required. This is in contrast to many other cases (e.g. the Incheon Tower, Abdelrazaq et al, 2011), where the design assumptions can be conservative.

The Merdeka 118 Tower was completed in 2023, and remains currently the 2nd-tallest building on earth at 678.9 m. The tower is shown in Figure 6.

Figure 6: Merdeka 118 Tower - completed

10 ACKNOWLEDGEMENTS

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