Design and verification of compression capacity of Continuous Flight Auger (CFA) Piles in Batesford Limestone for Surf Coast Highway Bridge

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ABSTRACT

The Surf Coast Highway (SCH) bridge is a vital rail bridge constructed as part of the South Geelong to Waurn Ponds Duplication (SGWP) project in Victoria, aimed at eliminating the at-grade crossing. The bridge is 105m long and four spans, with the abutment and piers supported on CFA piles. This paper presents a comprehensive discussion on the geotechnical model and interpreted design parameters for the underlying Alluvial Terrace and Batesford Limestone formation. The author examines the conventional methods of assessing pile capacity in accordance with the Australian Piling Code AS2159-2009, FHWA, and VicRoads standard specification Section 607. The approach encompasses a detailed analysis of stability, serviceability, and estimation of structural actions for design, considering uncertainties in ground conditions, constraints with varying pile spacings, and potential reductions in design parameters. A comparative analysis is also presented, contrasting the measured load capacity with the predicted pile capacity, providing valuable insights into the efficacy of the design and construction methodology employed.

Keywords: CFA Piles, pile testing, Batesford Limestone.

1 INTRODUCTION

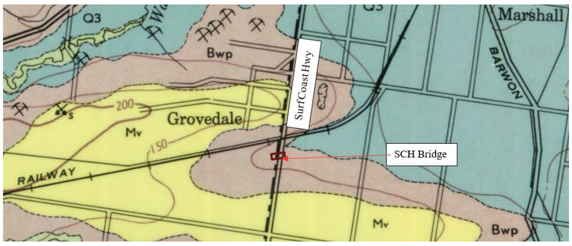
The South Geelong to Waurn Ponds (SGWP) project included duplication of 8 km of rail track, upgrading railway stations at South Geelong and Marshall and construction of new rail bridges crossing over Fyans Street and Surf Coast Highway. The project is delivered through Djilang Alliance comprising Rail Projects Victoria, McConnell Dowell, Downer, Arup and WSP. This paper discusses the design of the pile foundations for the Surf Coast Highway rail bridge. The piled foundations comprise a system of grouped continuous flight auger (CFA) piles, which are subject to a combination of vertical, lateral and overturning forces.

2 BRIDGE DESCRIPTION

The SCH bridge is a 105m long, four-span rail underbridge, spanning across the Surf Coast Highway (Figure 2). The bridge supports two ballasted tracks on separate superstructures. Each track is supported on a concrete U-Trough with maintenance and emergency egress walkways. The U-Troughs are supported by precast crossheads via elastomeric bearings. The crossheads are supported by cast in situ piers, pile caps and CFA piles. A minimum vertical clearance of 5.4m is provided over Surf Coast Highway.

3 GENERAL GEOLOGY AND SUBSURFACE CONDITIONS

The geology underlying the bridge location comprises thin layers of Quaternary aged Alluvial Terrace deposits, which can include minor river alluvium composed of stiff to hard clays. Underlying this is the Tertiary aged Sandringham sandstone comprising very stiff clay and outcrops of residual soil, composed of very stiff to hard clay from the Batesford Limestone, overlying Batesford Limestone residual soil and rock. Geology map (Geelong 1:63,360 Sheet) at the SCH bridge is shown in Figure 1.



Bwp – Batesford Limestone

Figure 1. SCH Location of Geology Map

The field investigation data at the bridge indicated presence of thin layer of fill layer directly overlying Alluvial Terrace (AT) which in turn was overlying Batesford Limestone (BL) which extends to depth greater than 30m. This is consistent with the anticipated geology. It is difficult to distinguish between the two formations, due to their similarities in material appearance, classification and strength.

4 DESIGN CHALLENGE

The geotechnical investigations revealed the presence of Limestone rock, which had undergone weathering to form residual soil and clay. The key challenge was to thoroughly characterise the clay's properties and strength to enable the design of floating piles.

However, a crucial step in ensuring foundation stability and safety pre-production pile load testing was not included in the project plan. Consequently, the necessary parameters were estimated through a combination of in-situ testing, empirical correlations, and a comparative analysis of various results.

This approach enabled the development of a robust foundation design, despite the absence of pile load test data.

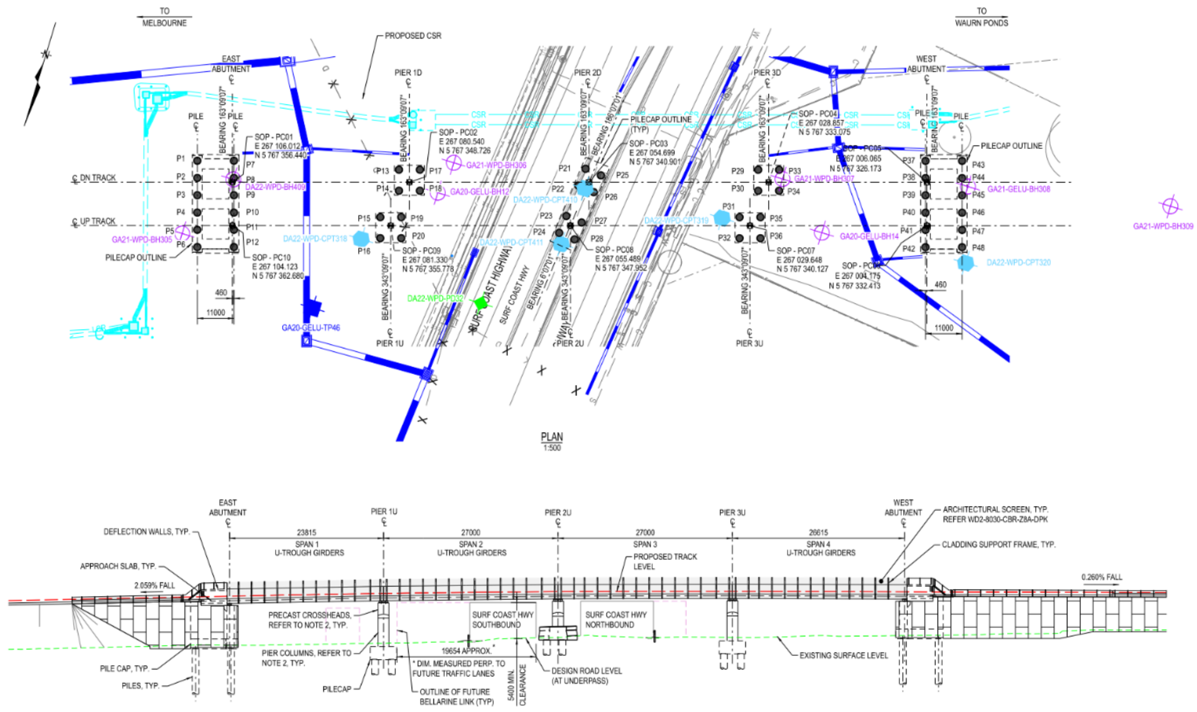


Figure 2. SCH bridge Foundation Plan and Elevation

5 PILE GROUP ARRANGEMENT

5.1 Pier

Pier 1 and 3 (UP and DOWN track) (Figure 3) are supported by individual pile cap (4.5 m x 4.5 m x 15m) and 4 no. 1,050mm CFA piles, and the pile cap is embedded 500mm below the ground surface.

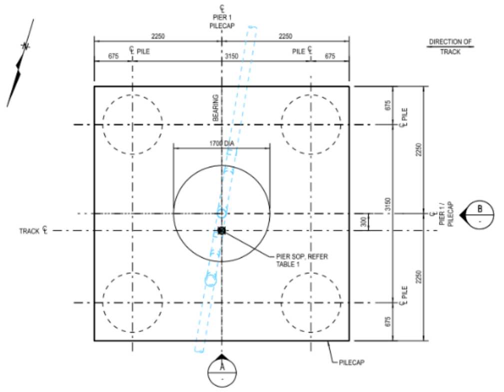


Figure 3. Typical; pile arrangement – Pier 1 & 3

Pier 2 (UP and DOWN) is supported by a pile cap which is structurally connected and supported by 4 no. CFA piles each. The pile cap is constructed above the ground surface.

5.2 Abutment

The abutment structure consists of forced cast-in-situ walls supported on a single pile cap. The walls support the front and side faces, supports the bridge bearings and the deflection wall adjacent to the rail. This is supported by two rows of 6 no. 1,050mm CFA piles (Figure 4).

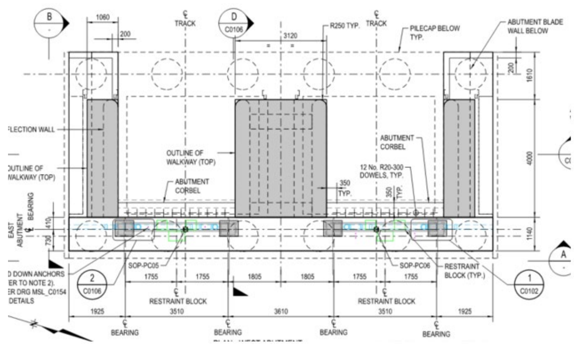


Figure 4. Abutments – pile arrangement

6 DESIGN STANDARD AND CRITERIA

The pile design was carried out in accordance with the provisions of AS2159-2009 Piling - Design and installation. The design ensured that the design geotechnical strength ($R_{d,g}$) was not less than the design action effect (E_d), which represents the factored load combination.

$$R_{d,g} = \phi_g R_{d,ug} \quad (1)$$

Where $R_{d,ug}$ design ultimate geotechnical strength and ϕ_g is the geotechnical strength reduction factor. The ϕ_g was estimated to be 0.73 for compression.

7 GROUND MODEL

A comprehensive ground model (Table 1) was developed for the abutments and piers, informed by the results of eight boreholes and five Cone Penetration Tests (CPTs), which were executed to a maximum depth of 44m.

Table 1. Ground Model

Geological Unit	Depth Range (m)	Material
Alluvial Terrace (Unit 1)	0 – 1.5	Clay, medium plasticity, Stiff to very stiff, MC close to PL
Alluvial Terrace/ Batesford Limestone (Unit 2)	1.5 – 40+	Clay and residual soil, high plasticity, very stiff to Hard, MC close to PL

The measured groundwater depth was 13.0m.

8 TESTING RESULTS

8.1 Borehole In-situ Testing

8.1.1 SPT Results with Depth

Figure 5 presents the corrected Standard Penetration Test (SPT) N(60) values for the Alluvial Terrace and Batesford Limestone materials, plotted as a function of depth. The undrained shear strength of the cohesive soils was estimated utilizing the SPT values, in accordance with the empirical equation (2) proposed by Hara et al. (1971) and Kulhawy and Mayne (1990). This approach is particularly applicable to clayey soils with moisture content approaching the plastic limit.

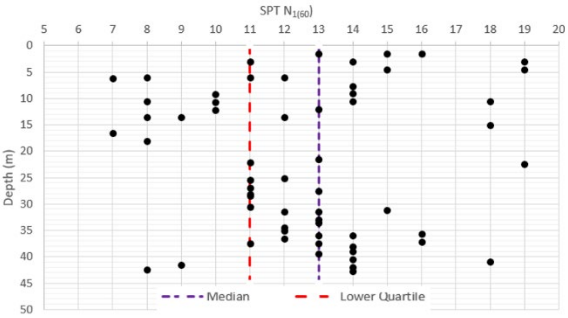


Figure 5. SPT N versus Depth(m)

$$S_u/P_a = 0.29\ N^{0.72} \qquad (2)$$

Where Pa is the standard atmospheric pressure equal to 101kPa. The estimated undrained shear strength with depth is shown in Figure 6.

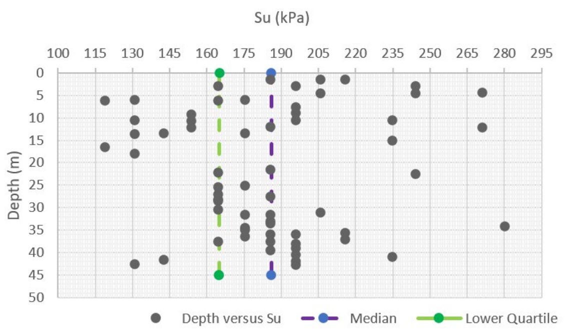


Figure 6. S_u (kPa) versus depth

8.1.2 Shear Vane (SV), Pocket Penetrometer (PP) Results

The shear strength of the undisturbed push tube samples was assessed through in-situ testing, utilizing a shear vane and pocket penetrometer. The resulting undrained shear strength profile, plotted against depth, is presented in Figure 7.

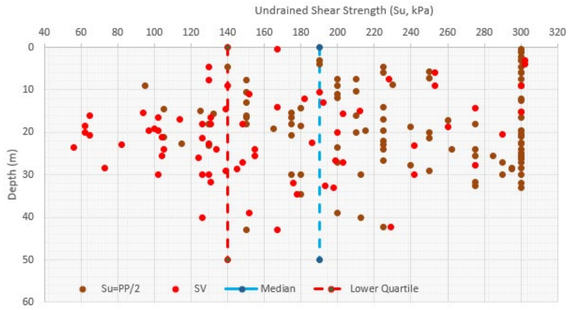


Figure 7. S_u (kPa) versus depth

A correction factor, μ, of 0.9 was applied to the shear vane test results, in accordance with the plasticity index values, as recommended by Bjerrum (1972). Additionally, the shear strength was estimated from the Pocket Penetrometer (PP) test results, using the empirical relationship: Su = 0.5 x PP.

8.2 Laboratory Test Data

Undrained shear strength (S_u) values were determined using unconsolidated undrained (UU) triaxial tests in the laboratory. The results at the bridge location with depth are shown in Figure 8.

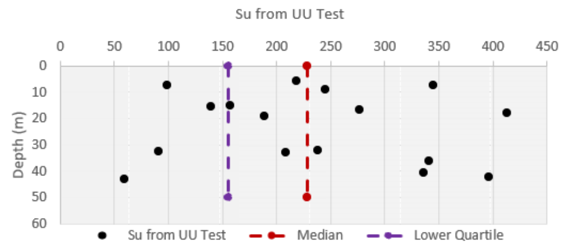


Figure 8. S_u (kPa) versus depth-UU Tests

8.3 CPT Results

The Cone Penetration Test (CPT) logs were interpreted to estimate the continuous profile of undrained shear strength with depth. The undrained shear strength (Su) values, expressed in kPa, were computed using equation (3):

$$S_u = (q_t - \sigma_v)/N_{kt} \qquad (3)$$

The N_{kt} value was calibrated using the results from the Shear Vane and Pocket Penetrometer tests, ensuring a reliable and accurate estimation of the undrained shear strength profile.

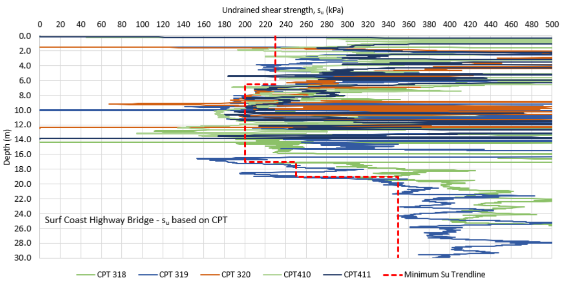


Figure 9. S_u (kPa) versus depth from CPT

Where q_t=corrected tip resistance; σ_v=overburden pressure; N_{kt}=bearing capacity factor ranged between 13 and 15.

9 INTEPRETATION OF RESULTS

The shear strength and deformation parameters were determined through a combination of in-situ and laboratory testing (Table 2). The interpretation of the design undrained shear strength, ultimate shaft friction, and end bearing resistance is presented in this paper, with the adopted design parameters for the Continuous Flight Auger (CFA) pile design summarized in Table 3.

A robust approach was taken to determine the design undrained shear strength, considering the lower quartile values derived from Standard Penetration Testing (SPT), Shear Vane, Pocket Penetrometer (PP), and Unconsolidated Undrained (UU) tests. Additionally, a weighted average value of the undrained shear strength over a 25-meter depth was adopted from the Cone Penetration Testing (CPT) results, with the data points ranging between 140 and 200kPa. This comprehensive approach ensured a reliable and accurate estimation of the soil's geotechnical properties, crucial for the design of the CFA pile foundation.

Table 2. Design Parameters (shear strength and deformation

Unit	ɣ (kN/m³)	C' (kPa)	Φ'	S_u (kPa)	v	E' (MPa
Unit 1	20	5	28	75	0.3	20
Unit 2	20	10	28	140 - 200	0.3	40

The shaft friction values were estimated using alpha (α) method as per FHWA (1999) method by O'Neil and Reese (1999).

$$\alpha=0.55 \quad S_u/P_a \leq 1.5; \tag{4}$$

Value of α varies linearly between 0.55 and 0.45 for $1.5 < S_u < S_u/P_a \leq 2.5$.

The α value ranged between 0.55 to 0.5 and the corresponding ultimate shaft friction ranged between 77 and 100kPa. A unit shaft friction within this range was adopted, accounting for the variation across the site and depth and also allowing for possible reduction in the strength due to long term creep.

Ultimate end bearing (q_p) is equal to $N_c^* S_u$; where N_c^*=9 for $S_u \geq$ 200kPa and N_c^*=4/3 [ln(Ir) +1] for S_u <200kPa

Where Ir is the rigidity index, equal to $E_s/3S_u$, where E_s is the undrained Young's modulus of soil just below the pile tip.

Table 3. Design Parameters -CFA Pile

Unit	Ultimate shaft friction, (kPa)	Ultimate End Bearing, (kPa)
Unit 1	40	675
Unit 2	80	1,800

10 PILE ANALYSIS

The behaviour of the pile group under axial, lateral, and torsional loadings was comprehensively analysed using the GROUP software (Ensoft Inc). This analysis enabled the determination of both p-y and T-q curves, which characterize the non-linear response of the soil to various loadings. Additionally, the software facilitated the calculation of the axial load distribution among individual piles within the group, providing valuable insights into the structural behaviour of the pile foundation system.

10.1 Load Combinations

The structural engineer developed and provided multiple load cases and combinations for the geotechnical analysis. The critical load case, which yielded the maximum ultimate axial loading, is summarized in Table 4. This load represents a combined force acting concentrically at the center and top of the pile cap, simulating the most severe loading scenario.

Table 4. Pier 1, 3 Critical Load Combination (ULS)

Axial Force, N* (kN)	Shear Force, V* (kN)		Bending Moment, M* (kNm)	
	Long	Trans	Long	Trans
11,500	820	15	5150	1,310

10.2 Abutment Analysis

Upon completion of the piles, the abutment approaches were backfilled, and a comprehensive soil-structure interaction analysis was conducted to assess the additional loadings resulting from settlement of the fill and subgrade, as well as soil pressures and applied forces. This analysis accounted for both immediate and long-term consolidation settlements, which were determined based on laboratory test results. Notably, the test results indicated that the soil was in the over-consolidation range for the applied loads, a finding that was incorporated into the analysis. To estimate the maximum forces and settlements in the pile, a finite element analysis was performed using Plaxis 3D (Figure 10). This rigorous approach enabled a detailed understanding of the soil-pile interaction and its impact on the structural behaviour of the abutment.

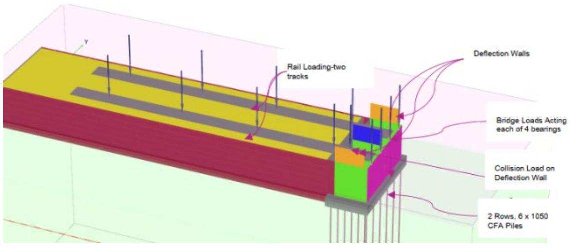


Figure 10. Abutment – Plaxis 3D Model

The resulting forces in the pile were subjected to load factors (Table 5) and Load combinations (Table 6) in accordance with AS5100.2-2017 Bridge Design, Part 2: Design Loads.

Table 5. Load Factors

Dead Load (DL) for walls, piles, abutment	1.2 or 0.9
Soil Loads	1.25
Bridge Loads	1.0 (for ultimate loads)
Rail Surcharge	1.5
Collision on deflection wall	1.0

Table 6. Load Combinations (AS5100.2-2017)

PE +DF	DF was estimated using FE analysis
PE + LL	Rail Loading
PE + LL + Collision	Unfactored bridge loads, i.e. LF=1.0 was adopted, as per AS5100.2-2017, as this produces a more severe loading
PE + DF + LL	As strength and stability as per AS5100.3 and AS5100.2

PE=permanent effect; LL=live load; DF=downdrag force due to settlement of soil.

10.3 Resulting Forces in Pile

The maximum axial loads computed on the piles for ultimate and serviceability load cases are summarised in Table 7.

Table 7. Maximum Axial Load (compression) on Pile

Bridge	Pile Location	ULS N* (kN) (+Comp / -Tension	SLS N (kN) (+Comp)
Both Abutments	Front Row	+6,000	+4,400
	Back Row	+4,500 / -1,500	+3,100
Piers 1 & 3	Selected maximum	+4,220	+2,705
Pier 2		+4,225	+2,930

10.4 Design Pile Length

The required pile length (Table 8) was determined in accordance with the design standard (Section 6), considering both geotechnical capacity and serviceability requirements.

Table 8. Design Pile Length

Bridge	Pile Location	Design Pile Length (m)	Depth of Pile cut of level
Abutment	Front Row	27.0	0.5
	Back Row	20.0	0.5
Pier 1 & 3	All Piles	21.0	2.0
Pier 2		21.0	0.1 – 0.4

11 PILE TESTING

Following construction of all the CFA piles as per VicRoads standard specification, Section 607 and AS2159-2009, both pile integrity tests (PIT) and high strain dynamic testing were carried out on the representative piles for abutments and piers. Two types of sensors namely, strain gauge and accelerometer were installed to record the forces and motion after each drop of hammer. Pile Driving Analyzer (PDA) was used for recording forces and motion. The pile load carrying capacity was analysed using computer program, CAPWAP based on the recorded data. The number of PDA tests carried out for the bridge and their location is summarised in Table 9.

Table 9. Pile Dynamic Testing (PDA) and Test Load

Bridge	Number of PDA Tests	Test Load, P_g (kN)
Abutment A and B	1 Test -Front Row	8,110
	1 Test – Back Row	6,085
Pier 1, 2, and 3	2 Tests, i.e. 1 per pile group	5,710

The acceptance criteria for the pile test, is based on limiting the pile head movement at P_g, to maximum deflection as given in equation (5).

$$P_g L/AE + 10 + 0.05\, d_t \quad (5)$$

Where L=length of pile in contact with ground; E=Young's modulus of concrete pile; d_t=pile base diameter; and A=area of the pile base.

The shaft resistance is mobilised with relatively small pile movement and is typically less than 10mm. The end bearing is mobilised with larger displacements, with typical tip movements in the range of 5–10 percent of the pile diameter.

11.1 Pile Testing Results

The results indicate the pile head movements to be less than 10mm, decreasing with depth and maximum tip movement of 6.6mm for the shortest pile and 4.0mm for the longest pile. A selection of four critical test results has been chosen for in-depth discussion in this paper, with a comprehensive summary of the test results presented in Table 10.

Table 10. Pile Dynamic Testing (PDA) Results

Pile Location	Installed Pile Length (m)	CAPWAP Capacity, kN		
		Total	Along Shaft	At Toe
P15 – Pier 1U	23	11,920	10,534	1,386
P1 – East Abutment-Back Row	21	12,500	11,700	800
P7 – East Abutment-Front Row	28	16,000	14,550	1,450
P13- Pier 1D	23	12,249	11,155	1,094

The end bearing resistance values obtained from the testing were found to be lower than the design values, suggesting that the end bearing capacity may not have been fully mobilized under the applied load or that the material at the pile tip was disturbed during pile construction, resulting in a reduced capacity. Consequently, the design end bearing resistance was not verified in the test.

The shaft resistance determined through CAPWAP analyses was compared to the design resistance and to the CPT and SPT testing results, utilizing correlations discussed in preceding sections of this paper. The distribution of shaft resistance in kN is presented in Figure 11.

Notably, the measured shaft capacity between 0–14m depth closely aligns with the shaft capacity derived from CPTs 410 and 411, while the capacity from CPTs 310 and 311 falls between the measured and designed capacity. Furthermore, the slope of the measured capacity for PDA Test at P15 and P13 below 14m depth is similar to the slope of the designed capacity, suggesting that the shaft friction may not have been fully mobilized at these depths.

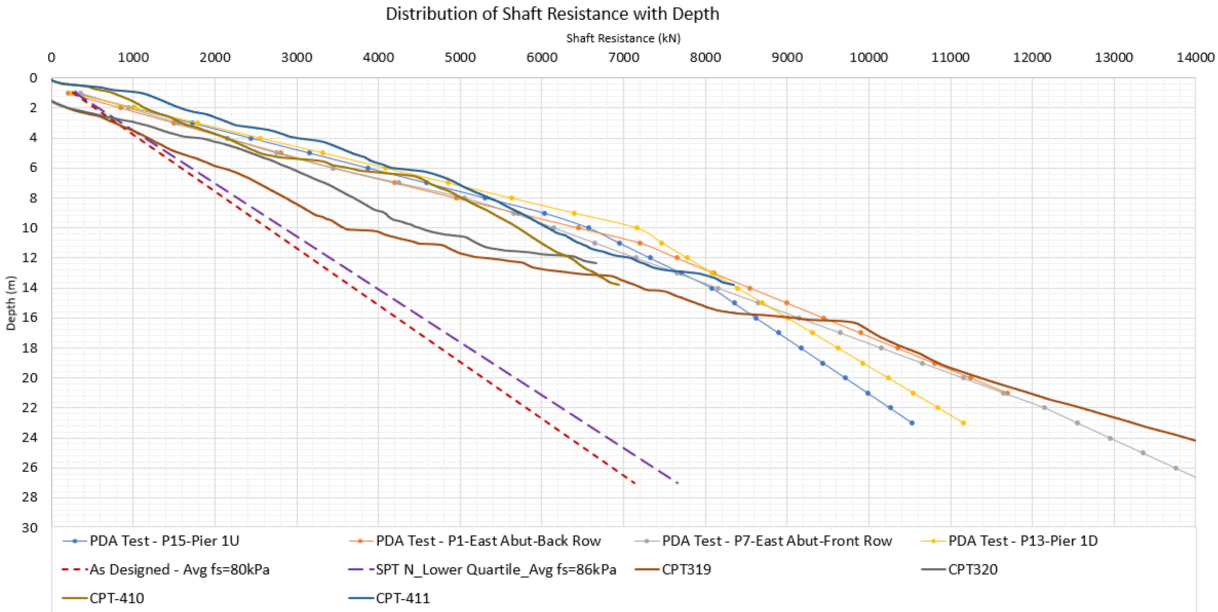


Figure 11. Distribution of Shaft Resistance with Depth

12 CONCLUSION

In summary, the pile dynamic load tests confirmed the geotechnical strength of the piles to be satisfactory, meeting the required standards. The discrepancies in the strength results obtained from in-situ testing and pile testing can be attributed to various factors such as variation of the ground conditions between location of the in-situ tests and the piles, difference in the diameter of the exploratory tests versus the pile diameter, variation in the moisture content. The author strongly advocates for the implementation of a comprehensive testing program for all CFA piling works. This program should encompass pre-production load testing and production testing. Pre-production testing involves the installation of test piles to establish construction methodology and load capacity. The results obtained can then be utilized to inform the design of production piles and ensure their installation meets acceptable limits. While the author acknowledges that pre-production pile testing may not always be feasible due to constraints in the construction program and associated additional costs, it is crucial to discuss this with the wider project team at the outset. This enables the identification of opportunities for initial testing and highlights the risks associated with inefficient or deficient engineering design. By doing so, potential risks can be mitigated, and the overall success of the project can be ensured.

13 ACKNOWLEDGEMENT

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