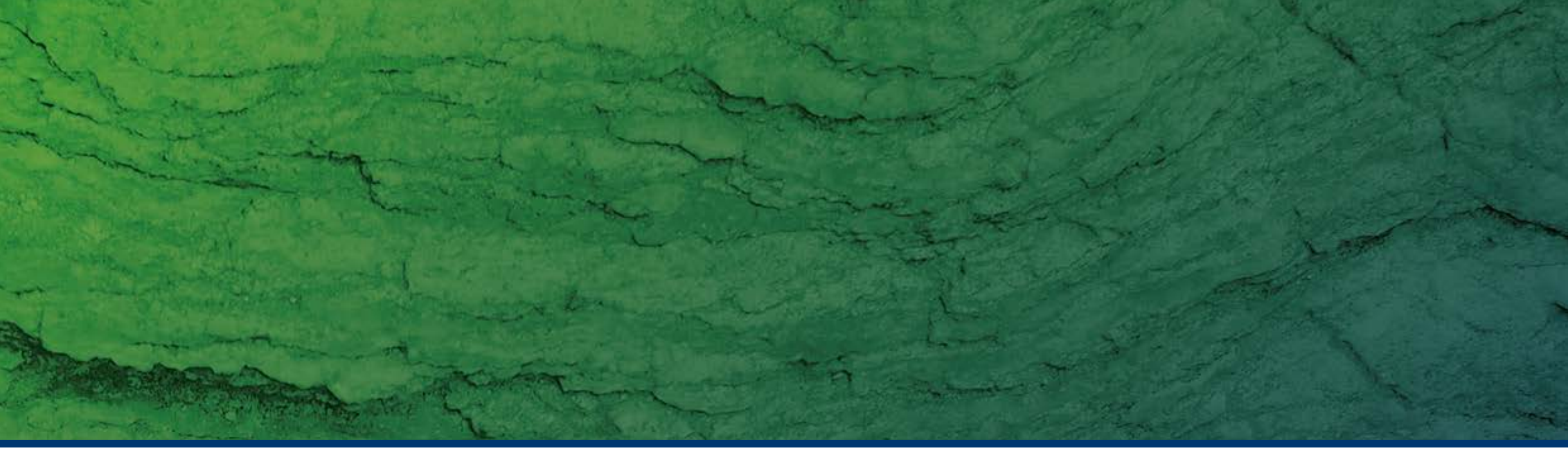
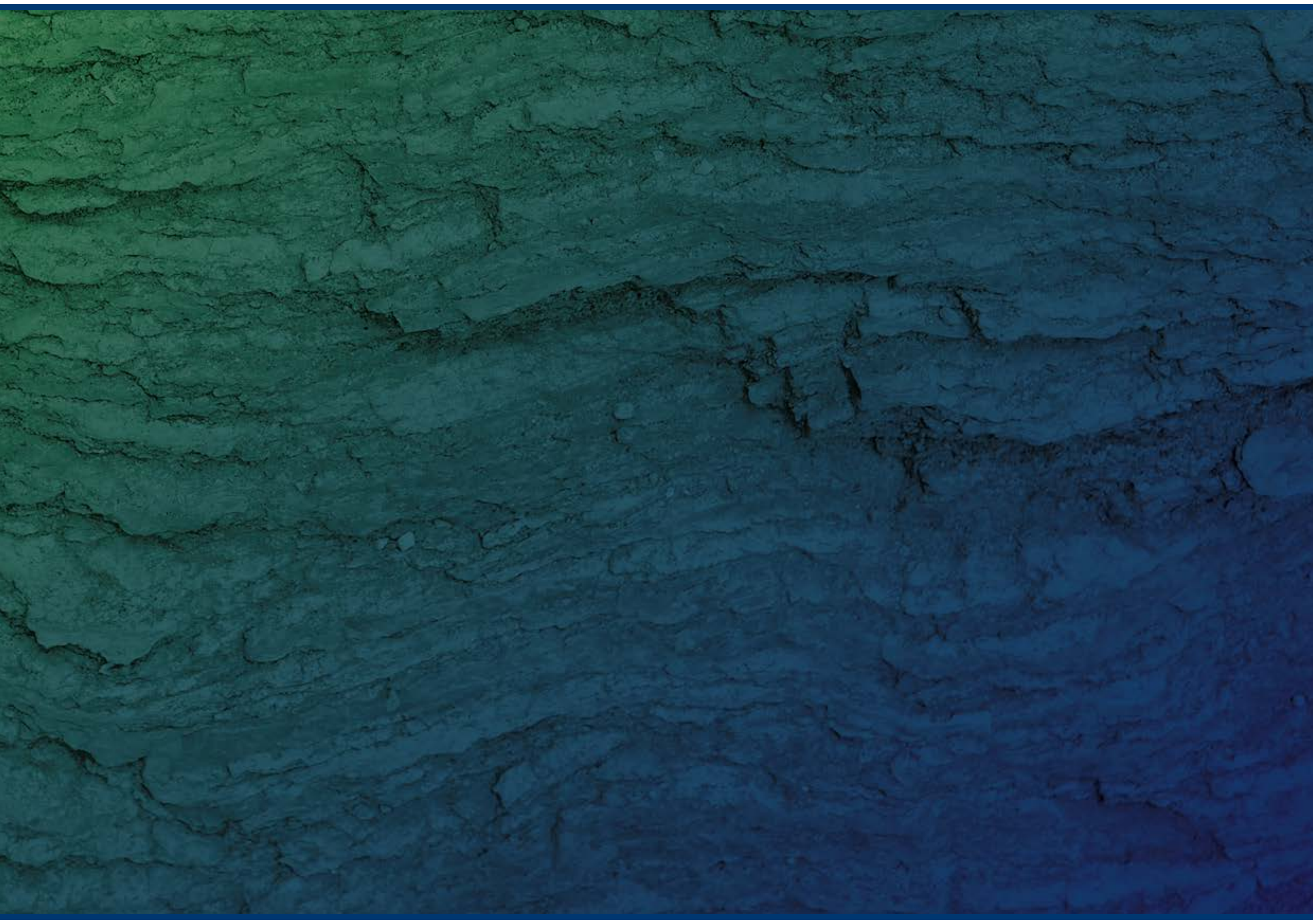


PROCEEDINGS

2019 AUSTRALIAN GEOMECHANICS SOCIETY VICTORIAN SYMPOSIUM

Geotechnical characterisation – managing design and construction risk

Wednesday, 30 October 2019, 8:00am – 7:00pm
Rydges Hotel, 186 Exhibition Street, Melbourne



AUSTRALIAN GEOMECHANICS SOCIETY
VICTORIA CHAPTER

reducing geotechnical uncertainty
IGS
Insitu Geotech Services Pty Ltd

PREFACE

The Victorian chapter of the Australian Geomechanics Society invited academics and practitioners in the field of geotechnical and ground engineering to attend the 2019 Australian Geomechanics Society Victorian Symposium held on 30 October 2019.

In recent years Victoria has seen significant growth in the construction industry. Investment in both public infrastructure and commercial real estate is growing, and as our cities and infrastructure grow, so too does the need to develop parcels of land with challenging ground conditions. Economical and safe geotechnical design requires efficient and well thought through ground investigation and characterisation to identify and manage ground risks and opportunities.

The 2019 Australian Geomechanics Society Victorian Symposium presents an overview of current state of-the-art practices, innovation, new research results and case studies relating to geotechnical characterisation with an emphasis on its implications for addressing and managing design and construction risk. The 2019 Symposium brought together professional engineers, researchers, specialist contractors, regulators, educators and students to share and discuss their experiences on the topic of ground characterisation.

ORGANISING COMMITTEE*

Daniel King (Co-Chair)

Ross Kristinof (Co-Chair)

Andrew Lochaden

Mike Shackleton

*a sub-committee of the AGS Victoria committee

SPONSORS

Platinum

Insitu Geotech Services (IGS)

Gold

Chadwick Geotechnics

Global Synthetics

Silver

Civil Geotechnical Services (CGS)

Black Insitu Testing (BIT)

GBG Group

Insitu Test

Probedrill Geotechnical Survey

TECHNICAL REVIEWERS

All technical papers in these proceedings, excluding the keynote addresses, were peer reviewed. The reviewers are acknowledged and listed below:

Daniel King (Editor)

Greg Anderson

Stephen Durham

Ross Kristinof

Jay Lee

Robert May

Guillermo Narsilio

Bhavikh Riyat

Sergei Terzaghi

Bhupatindra Vaidya

A technical society of

AUSTRALIAN GEOMECHANICS SOCIETY

ENGINEERS AUSTRALIA

Application of pile test data for geotechnical site characterisation of Seaford and Carrum

N. M. Withers[1], S. Visvalingam[2] and C. Mc Colgan[3]

[1]WSP Australia, 28 Freshwater Place, Southbank VIC 3006; PH (03) 9412 5100; email: Nick.Withers@wsp.com
[2]email: Suthagaran.Visvalingam@wsp.com, [3]email: Cillian.McColgan@wsp.com

ABSTRACT

Victoria's Level Crossing Removal Project is removing 75 level crossings across Melbourne to improve safety, reduce congestion and allow more trains to run more often. The Southern Program Alliance (SPA) is delivering the Frankston line section – a $3 billion upgrade removing 18 level crossings and building 12 new stations. The Seaford level crossing removal works are complete, and those in Carrum, Mentone and Cheltenham are under construction.

At Seaford, dynamic pile testing analysis of continuous flight auger (CFA) pile foundations and rigid inclusions, using pile driving analyser (PDA) testing and Case Pile Wave Analysis Program (CAPWAP) analysis, was carried out to validate pile design assumptions. The results were correlated to in-situ and laboratory soil classification testing to characterise the site's geology in terms of pile performance. This enabled the testing results to be used to refine the geotechnical design of CFA pile foundations for the Carrum rail viaduct, about two kilometres north of Seaford, in the same geology.

This paper discusses characterisation of geological units encountered at Seaford using pile test data, and the subsequent geotechnical design of the Carrum rail viaduct foundations. Comparing soil classification tests from Seaford to Carrum showed good agreement, and satisfied the designers that the Seaford pile test data was applicable for use in the foundation design at Carrum. Furthermore, this paper compares the correlation of geotechnical strength parameters with available empirical correlations, and comments on the benefits and limitations of this site classification and design approach.

Keywords: CAPWAP analysis, CFA piles, site characterisation.

1 INTRODUCTION

Removing level crossings improves safety by separating trains from traffic, while also reducing congestion, upgrading the road network, connecting communities and supporting urban regeneration. The Victorian State Government initiated a level crossing removal program to remove at least 75 level crossings across Melbourne by 2025. The Southern Program Alliance (SPA) comprises WSP, Acciona, Coleman Rail, Lendlease, Metro Trains Melbourne (MTM) and the Level Crossing Removal Project (LXRP). The Alliance is delivering level crossing removal works in Seaford (complete), Carrum, Mentone and Cheltenham (under construction).

The Seaford Road bridge is a rail-over-road solution to removing the level crossing at Seaford Road. The structure has two parallel U-trough girder bridges, supported by four abutments and four piers, which are founded on continuous flight auger (CFA) piled foundations. Geotechnical investigations revealed soil conditions consistent with much of South-East Melbourne – Quaternary deposits underlain by Tertiary aged Sandringham Sandstone (formerly Brighton Group), and Gellibrand Marl (formerly Fyansford Formation). The Seaford Road bridge CFA foundations were designed using conventional empirical methods.

The Carrum rail viaduct, 2km north of Seaford, is also a rail-over-road solution in similar conditions. The bridge is supported on two abutments with 36 piers on CFA pile foundations. High-strain dynamic pile tests on the Seaford Road bridge production piles used pile driving analyser (PDA) and Case Pile Wave Analysis Program (CAPWAP). Results revealed axial capacity exceeded design capacity, which presented an opportunity to refine the design for the Carrum bridge foundations, to be constructed in similar geological conditions. The number of piles to be installed at Carrum meant significant potential savings if greater capacity could be proved from sacrificial piles at Seaford. Additional destructive pile tests investigated the performance of piles at the Seaford site under higher test loads and founded at shallower depths.

The designers interpreted and correlated the CAPWAP analysis outputs, pile installation logs, and on-board pile installation records with in-situ and laboratory soil classification tests from Seaford. Good agreement was found between geological units and the pile test data, and geological units were assigned unit shaft friction and end-bearing parameters (reported in more detail in Mc Colgan et. al.,2019). Comparison between soil classification tests at Seaford and Carrum satisfied the designers that adopting the site-specific empirical unit pile parameters was an appropriate method of assessing design axial pile capacity.

This paper discusses the characterisation of geological units encountered at Seaford using pile test data, and its use in designing the Carrum rail viaduct foundations. The results of this assessment are also compared to existing empirical correlations, and limitations are discussed.

2 SEAFORD ROAD BRIDGE

2.1 Structure description

The Seaford Road bridge includes a rail-over-road bridge and a pedestrian bridge consisting of U-trough girders with a total combined length of 111 m. The bridge foundations are founded on groups of 1.05 m diameter CFA piles to varied depths. The pile group comprises eight and 12 piles at piers and abutments respectively. The bridge approach embankments are founded on 0.45 m and 0.6 m diameter controlled modulus columns (CMC) to limit ground movements at the abutments.

2.2 Geology

Seaford Road bridge is in the Brighton coastal plains, more specifically in the Carrum sunk lands, bounded by the Beaumaris Monocline to the north, the Melbourne Warp to the east, and the Selwyn Fault to the south. The site is approximately located at Selwyn Fault. The ground profile in this area generally comprises marine tertiary sediments, overlain by quaternary swamp and sand deposits (Kenley, 1992; Gronvall, 1992).

The geotechnical investigations included 19 boreholes to a 46.85 m maximum depth below existing ground level (begl), and 25 Cone Penetration Tests (CPTu).

2.3 Pile design

The Seaford Road bridge pile design accords with AS2159-2009 "Piling – Design and installation". Geotechnical soil unit parameters were assigned based on in-situ and laboratory tests using empirical correlations from Poulos (1997) and Look (2007).

The unit shaft friction and end bearing values presented were limited to the values Decourt (1995) recommended. The average skin friction was assessed based on O'Neill & Reese (1999) for cohesive soils and Craig (2004) for granular soils. The ultimate bearing capacity was assessed based on Fleming et al. (2009) for cohesive soil and Berezantev (1961) for granular soils.

An adopted geotechnical reduction factor of 0.72 was based on the standards mentioned, and tested in accordance with VicRoads 607.

The resulting foundation design required all piles to be founded in the tertiary Gellibrand Marl formation.

2.4 Pile testing

A purpose-built test frame with a 12T drop hammer was used to test the nominated production piles. Drop

heights varied from 0.8 to 1.5m, correlating to applied test energies of between 94 and 176 kJ.

All piles were shown to achieve the design loads with some reserve capacity. Mc Colgan et al, (2018) provides a more detailed description of the pile test procedure and test results analysis. Table 1 shows the results of the measured shaft friction and end bearing from the Seaford pile tests. The positive test results led to investigating whether this data was applicable for use in similar geological conditions.

3 CARRUM RAIL VIADUCT

3.1 Structure description

The Carrum rail viaduct is a 19-span, 545 m long bridge with reinforced concrete U-trough girders. The rail viaduct supports the new Carrum Station and spans over McLeod Road. The pier and abutment foundations use 1.2 m diameter CFA pile groups, with four and six piles installed to varied depths to support the piers and abutments, respectively.

3.2 Geology

The ground conditions encountered at Carrum are consistent with the available regional geological information, and are similar to those at Seaford, except that the site is north of the Selwyn Fault in the Carrum sunk lands, and closer than Seaford to the existing shoreline. This results in a deeper profile of tertiary marine sediments, overlain by a greater depth of quaternary swamp and sand deposits(Kenley, 1992; Gronvall, 1992).

The geotechnical site investigations included 35 boreholes to a maximum depth of 49.65 m begl, and 46 CPTu tests.

4 COMPARISON OF SEAFORD AND CARRUM

4.1 Methodology

We assessed in detail the geological units at both Carrum and Seaford to determine the feasibility of using the Seaford bridge pile test data in the Carrum rail viaduct design.

4.2 Geology

Broadly, the geology encountered at Seaford and Carrum are similar. Both sites are part of the Carrum sunk lands and both were expected to have deep deposits of tertiary marine sediments of the Sandringham Sandstone and Gellibrand Marl formations, overlain by surficial quaternary swamp and sand deposits. Figure 1 provides a modified excerpt of the Geological Survey of Victoria 1:25,000 geological map of Chelsea (Tickell et al.,1980) and Table 2 presents an interpretation of the geological units encountered at both Carrum and Seaford.

Carrum is slightly higher in elevation than Seaford, with the typical surface reduced level (RL) being 6 m Australian height datum (AHD) to 3 m AHD respectively. As Seaford's distance from the shoreline is further than

Table 1. Interpreted pile test results at Seaford, with Standard Penetration Test (SPT) correlation

Geological Unit ID	Unit skin friction (kPa)			Unit End Bearing (kPa)		
	Adopted $N_{1,60}$ [a]	Shaft Friction Range	Average	Adopted $N_{1,60}$ [b]	End Bearing Range	Average
Unit 2	15	50 – 80	65	N/A	N/A	N/A
Unit 3A	23	50 – 120	80	26	2000 – 6000	4300
Unit 3C/3D [c, d]	33	80 – 150	100	30	4750 – 6500	5900
Unit 4	21	100 – 150	120	21	1500 – 4000	2300

[a] Refer Table 2 for description of geological unit ID.
[b] SPT $N_{1,60}$ used for shaft friction is adopted based on all corrected SPT values undertaken within unit.
[c] SPT $N_{1,60}$ used for end bearing includes SPT values from 2 diameters of pile length above the pile toe, and 3 diameters of pile length below the toe.
[d] Notation of units skips the letter "B", due to comparison with Carrum geological units.
[e] Units were considered as one sub-formation at the time of design, and were subsequently differentiated.

that of Carrum, it was expected that there would be less dune sands (Unit 2A) overlying the swamp deposits at Seaford. This would also explain why the beach sands (Unit 2B) were not encountered at Seaford. Note that in Carrum a material was encountered that was not reported in regional geological information (Unit 2D). Underlying the quaternary swamp deposits, is a mix of grey to brown clayey sand and sandy clays with generally medium dense, or soft-to-stiff consistency. Shell fragments were also encountered in this material. It is possible this material originated from weathering of the Sandringham Sandstone upstream being subsequently re-deposited in the quaternary era. It is noted that Unit 2D was not identified at Seaford. The CPT logs show some evidence of this layer, but it was not encountered in the boreholes.

Seaford is at the approximate Selwyn Fault location (Figure 1). This may be the reason for the shallower maximum depths to the base of the Sandringham Sandstone, and smaller maximum unit thickness when compared to Carrum. Both Units 3A and Unit 3C showed good agreement in logged description between the sites. The predominantly clay layer within the Sandringham Sandstone (Unit 3B) encountered at Carrum was not found at Seaford. This unit's absence may also be due to the Selwyn Fault presence. Materials below Unit 3C were not proposed as a founding stratum for Carrum bridge and thus these were not investigated in detail.

The Gellibrand Marl unit was encountered at far shallower depths at Seaford than at Carrum, which again may be due to the Selwyn Fault. Similar borehole descriptions were logged at both sites. Again, this material was not assessed in detail due to the large depth.

4.3 In-situ tests

While the similar geological origin of the materials at Carrum to those at Seaford was an important factor in adopting Seaford pile test data, in-situ strength testing was the primary assessment of applicability.
Field SPT values were corrected for energy and overburden stress (for granular soils), using the methodology presented by Skempton (1986) to achieve $N_{1,60}$ values. The SPT $N_{1,60}$ values generally show good agreement between the units encountered at Seaford and Carrum. More specifically, the materials at Carrum generally show higher penetration resistance than those at Seaford. Table 3 presents a summary of the $N_{1,60}$ values for each geological unit encountered at both sites.

SPT penetration resistance in the quaternary dune sands (Unit 2A) were very similar between the two sites. Unit 2B, while not encountered at Seaford, showed high penetration resistance and was generally quite coarse. This would indicate that the material should theoretically lie towards the upper end of the range of the granular materials encountered at Seaford in terms of shaft friction capacity. Unit 2C and 2D were not differentiated at Seaford so it was difficult to compare in-situ testing.

The Sandringham Sandstone and Gellibrand Marl units are present at both sites, and higher penetration resistance was consistently recorded at Carrum. The presence of relatively low $N_{1,60}$ values for Unit 3B at Carrum was also considered a potential issue when correlating the sites for design consideration.

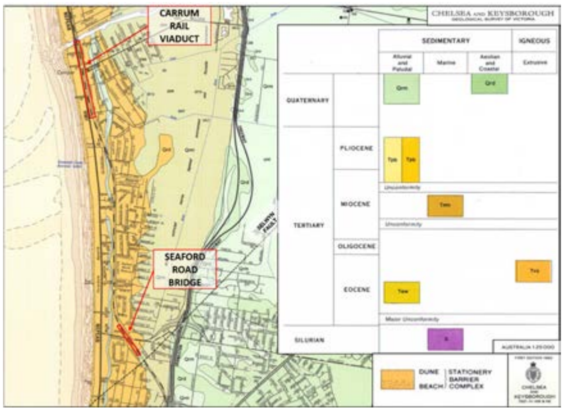


Figure 1. Modified excerpt of 1: 25,000 Chelsea map (Tickell et al., 1980)

4.4 Laboratory tests

The soil index properties of particle size distribution (PSD) and Atterberg Limits generally show good agreement between the units encountered at Seaford and Carrum. Table 4 presents a summary of the representative values of gravel, sand and fines fraction, and Atterberg limits for each geological unit at both sites.

The grading of Unit 2A is almost identical between the sites, with most sand falling under the medium grained size (0.3 to 0.6 μm). The grading of Unit 2C shows a higher percentage of fines at Carrum, with a corresponding increase in the material's plasticity. This was also observed in the logs, with an increase in dark grey organic clays.

The grading and plasticity of Unit 3A is again almost identical between the sites. The Unit 3C grading shows a similar percentage of fines and granular material. However, there appeared to be more cementation at Seaford than at Carrum, with the presence of predominantly medium-grained gravel-sized nodules frequently recorded on the borehole logs. It is further noted that the plasticity of this unit at Carrum is higher than at Seaford.

There is good agreement between each site's Unit 4 gradings – both sites exhibited greater than 80% of fines material. The material at Seaford is of high plasticity, while the material at Carrum is of medium plasticity.

5 ADOPTED DESIGN METHODOLOGY

Based on the comparison, the designers assessed that the Seaford bridge pile test data was a sound basis for the Carrum rail viaduct design, and used the following methodology when designing the Carrum piles:

Table 2. Geological Profile of Seaford and Carrum

Geological unit ID	Material (General description)	Seaford – Top of unit level (m AHD)	Carrum – Top of unit level (m AHD)
Unit 1	Fill *(Shallow fill associated with rail)*	4.5 to 2.1	6.0
Unit 2A	Quaternary Sand Deposits *(Dune sands; fine to medium grained, brown, generally medium dense)*	4.4 to 2.0	6.0 to 5.5
Unit 2B	Quaternary Beach Deposits *(Fine to coarse sands with shell fragments; grey to pale grey, generally dense to very dense)*	Not encountered	1.0 to 0.5
Unit 2C	Quaternary Swamp Deposits *(Fine grained silty sands with occasional layers of organic, compressible clays, dark grey, generally loose or very soft)*	0.5 -to -0.5	-3.0 to -4.5
Unit 2D	Quaternary Alluvial Deposits *(Fine to medium grained sands with shells and sandy clays, grey, brown, generally medium dense or soft to stiff)*	Not encountered	-5.5 to -6.5
Unit 3A	Tertiary Sandringham Sandstone – Interbedded Sands and Clays *(Sandy clays of medium plasticity and fine to medium grained sands; grey, brown, generally medium dense or stiff)*	-2.5 to -5.0	-6.5 to -9.0
Unit 3B	Tertiary Sandringham Sandstone – Clay *(High plasticity clay with sand; grey, dark grey, generally stiff, occasional firm)*	Not encountered	-13.5 to -14.0
Unit 3C	Tertiary Sandringham Sandstone – Clayey Sand *(Clayey and silty sands, with occasional clayey sand bands and cemented bands; pale brown, pale grey, sometimes red brown, orange-brown, generally dense)*	-6.0 to -8.0	-16.5 to -19.0
Unit 3D	Tertiary Sandringham Sandstone – Clayey Sand / Sandy Clay *(Fine grained clayey and silty sands, with cemented nodules; brown, orange-brown, generally medium dense to dense)*	-7.0 to -12.5	Not encountered
Unit 4	Tertiary Gellibrand Marl *(Clayey Silt and silty clay of medium to high plasticity; grey, green-grey, generally very stiff to hard)*	-15.0 to -19.0	-27.0 to -30.0

Table 3. Soil classification results per unit at Carrum

Geological unit ID	Carrum $N_{1,60}$ values				Seaford $N_{1,60}$ values			
	Number of tests	Average	Lowest	Highest	Number of tests	Average	Lowest	Highest
Unit 2A	76	10	4	24	32	16	4	43
Unit 2B	83	43	12	75	-	-	-	-
Unit 2C	41	8	0	34	6	5	0	14
Unit 2D	34	18	0	65	-	-	-	-
Unit 3A	96	34	1	75	23	23	10	71
Unit 3B	29	9	2	18	-	-	-	-
Unit 3C [a]	123	39	8	75	49	33	0	70
Unit 3D [a]	-	-	-	-				
Unit 4	64	35	4	75	68	21	10	65

[a] Unit was not differentiated at Seaford to reflect design assumptions as no major difference in values between units was discernible.

Table 4. Soil classification results per unit at Carrum and Seaford

Geological unit ID	Gravel content (%)	Sand content (%)	Fines content (%)	Limit liquid (%)	Plastic limit (%)	Plasticity index
Unit 2A	1 (1)	98 (97)	2 (2)	- (-)	- (-)	- (-)
Unit 2B	0 (n/a[b])	93 (n/a)	7 (n/a)	- (n/a)	- (n/a)	- (n/a)
Unit 2C	0 (0)	75 (88)	25 (12)	45 (33)	19 (21)	26 (12)
Unit 2D	0 (n/a)	72 (n/a)	28 (n/a)	44 (n/a)	18 (n/a)	26 (n/a)
Unit 3A	0 (1)	58 (59)	42 (40)	47 (44)	20 (19)	27 (26)
Unit 3B	0 (n/a)	34 (n/a)	66 (n/a)	64 (n/a)	22 (n/a)	42 (n/a)
Unit 3C	0 (20)	73 (50)	27 (30)	44 (26)	18 (16)	26 (10)
Unit 3D [a]	n/a (17)	n/a (40)	n/a (43)	n/a (35)	n/a (19)	n/a (17)
Unit 4	0 (0)	20 (7)	80 (93)	45 (61)	26 (27)	19 (35)

[a] Values of testing from Seaford presented in brackets
[b] Unit not encountered in specified site

Where units are encountered at both Seaford and Carrum:

- The lower bound to the average of the unit's mobilised shaft friction was adopted, based on the $N_{1,60}$ values of the closest boreholes. This resulted in the average values generally being adopted.
- The lower bound to the lower quartile of mobilised end bearing of the unit was adopted, based on the $N_{1,60}$ values of the closest boreholes.
- For Unit 2B, the upper bound to median of the data for Unit 3C was adopted due to the consistently high penetration resistance recorded.
- Where units are only encountered at Carrum:
    - Unit parameters were based on the empirical correlations stated in the Seaford pile design methodology.

Based on the above approach, piles were generally founded in Unit 3A, with a small percentage of piles founded in Unit 3B at the Southern end of the alignment. Piles were extended to Unit 3B where strength parameters inferred from $N_{1,60}$ values were less than the average encountered over the entire alignment.

The designers adopted a less aggressive approach to the unit end bearing for the pile foundations, as this was not consistently mobilised in the pile testing carried out at Seaford. Furthermore, the relatively high shaft friction adopted was enough to achieve a significant reduction in the pile foundation lengths.

5.1 Comparison of conventional correlations

Table 5 provides a simple comparison of the adopted design parameters at Carrum to conventional correlation parameters. This comparison is based on the entire site's generalised soil profile.

The adopted design approach meant a significant increase in unit parameters was observed. The higher shaft friction and end bearing available in Unit 3A of the adopted approach allowed most pile foundations to be founded above the Unit 3B layer, at a depth great enough to resist punching shear. Conversely, the conventional approach required at least three pile diameters embedment into Unit 3C before achieving the design load.

6 RISKS FOR CONSIDERATION

The authors acknowledge that there is risk in this design approach, specifically in the increased chance of pile failure from a statistical perspective. Designing piles to the average of empirical test data can theoretically result in an increased likelihood of piles failing under production testing. The Carrum viaduct is being constructed under significant time restraints, so this risk should be clearly balanced against potential cost savings. In this example the designers managed the risk in four ways:

- The design adopted less end bearing than would be expected for the material encountered at Carrum;
- Piling contractors were required to mobilise considerable test energy (double that used at Seaford) as part of the contract documentation to help mobilise end bearing if required;
- The construction team were involved in developing the design approach, and contributed to the risk-based decision; and
- A larger pile diameter (1200 mm) was adopted in conjunction with generally shallower piles, as opposed to deeper piles of a lower diameter. This gave the alliance the flexibility to adopt deeper piles if issues with pile testing were encountered.

When adopting these results, the actual diameter of the installed pile against the design diameter should be further considered. Due to the CFA piles' installation process, the need to maintain a constant concrete pressure upon auger withdrawal means it is common for the pile diameter to be greater than the auger diameter. Thus, care should be taken when appropriating the results of PDA tests and CAPWAP analysis to CFA piles of differing diameters. In the Seaford case, test data included piles of 0.45, 0.6 and 1.05 m diameter.

Furthermore, quality control of the pile installation procedure was proposed in the form of both on-site inspections of selected piles to confirm site conditions were consistent with the design ground model, and subsequent review of the on-board computer pile installation records.

Notwithstanding the above, the most robust risk mitigation would be to undertake pre-production tests at the site. This was considered during design development and ruled out due to the significant cost associated with the site constraints.

7 OPPORTUNITIES

The clear benefit of using such a design approach is the cost and time saving from reducing pile lengths. While it required an initial expenditure for sacrificial piles, there was significant saving of concrete volume and steel reinforcement over 160 piles. Furthermore, this increases the production of piling rigs and the associated labour costs, which adds to the productivity of the construction sequence and allows the superstructure to be completed earlier.

From a design perspective, it is beneficial for the industry to understand the behaviour of CFA piles within the Sandringham Sandstone and the Gellibrand Marl units. With more of this data available, CFA piles design in South-East Melbourne may potentially be refined. The result may be a decrease in construction costs for other major infrastructure projects.

Table 5. Comparison of adopted unit parameters with conventional parameters for CFA pile design at Carrum

Geological unit ID	Unit skin friction (kPa)		Unit end bearing (kPa)	
	Adopted design	Conventional	Adopted design	Conventional
Unit 2A	65	15	n/a	n/a
Unit 2B	110	65	n/a	n/a
Unit 2C	30 to 40	15	n/a	n/a
Unit 2D	30 to 50	35	n/a	n/a
Unit 3A	40 to 75	45	500 to 2500	2500
Unit 3B	40 to 50	40	750 to 1150	550 to 1150
Unit 3C	50 to 110	50	1150 to 3500	3500
Unit 4	120	40	2500	1150

8 CONCLUSION

In summary, the CFA pile design of the Seaford Road bridge foundations and associated pile testing regime, allowed the geotechnical design of the Carrum rail viaduct foundations to be refined.

PDA testing at a frequency of one per substructure was used to increase design confidence, and verify the construction quality. The pile test data acquired from Seaford showed significantly larger capacity than typical empirical correlations would yield, especially within the Sandringham Sandstone and Gellibrand Marl formations. This supports the approach adopted by SPA to undertake pre-production pile tests.

Geotechnical classification tests and in-situ strength tests, coupled with pile test data, can be used effectively to characterise the performance of a geological unit as a founding medium.

Once the pile installation at Carrum is complete, SPA should realise the significant cost and time benefits of this approach. It is the authors' opinion that geotechnical engineers should more thoroughly pursue pile testing. The industry can benefit from understanding local geology, and can realise significant cost savings over a major infrastructure project's life.

At the time this paper was submitted, pile installation at Carrum was underway. Initial pile test results suggest greater capacity than anticipated in design, further demonstrating the validity of the adopted design approach.

9 ACKNOWLEDGEMENTS

The authors would like to thank the Level Crossing Removal Project and the Southern Program Alliance for permission to present this work, and thank Richard Kaser and Paul Hewitt for their assistance in preparing this paper.

REFERENCES

Berezantzav, V.G. et al. (1961). "Load bearing capacity and deformation of piled foundations". Proceedings of 5th International Conference SMFE, Paris, Vol. 2, pp. 11-15.
Craig, R.F. (2004). "Craig's soil mechanics, 7th Edition". Spon Press, Taylor & Francis Group, New York, pp. 313-315.
Decourt, L. (1995). "Prediction of load – settlement relationship for foundations on the basis of SPT". Ciclo de Conferencias International, Leonardo Zeevaert, UNAM, Mexico, pp. 85-104.
Fleming W.G.K. (1992). "A new method for single pile settlement prediction and analysis". Geotechnique 42, No. 3, pp. 411-425.
Gronvall, G.S. (1992). "Engineering geology of Melbourne: The Fyansford Formation". Balkema, Rotterdam, pp. 175-189.
Kenley, P.R. (1992). "Engineering geology of Melbourne: Brighton Group - Geology". Balkema, Rotterdam, pp. 191-196.
Look, B. (2007), "Handbook of geotechnical investigations and design tables". Taylor and Francis Group, London.
Mc Colgan, C. et. Al. (2019). "Estimated versus measured capacity of CFA piles for Seaford Road bridges, Melbourne". Proceedings of 9th Australian Small Bridges Conference, Gold Coast, pp. 1-14
O'Neill, M.W. and Reese, L.C. (199). "Drilled shafts: construction procedures and design methods". Publication Number FHWA-IF-99-025, Federal Highway Administration, Washington D.C., pp. 758.
Poulos, H.G. and Davis, E.H. (1980). "Pile foundation analysis and design". John Wiley, New York, pp. 31.
Skempton, A.W. (1986). "Standard Penetration Test procedures and the effects in sands of overburden pressure, relative density, particle size, ageing and overconsolidation". Geotechnique 36, No. 3, pp. 435-447.
Tickell, S.J. et al. (1980), "Chelsea and Keysborough map". Geological Survey of Victoria, Sheets 7921-IV-NW & NE, Departments of Minerals and Energy, Melbourne.

2019 AUSTRALIAN GEOMECHANICS SOCIETY
VICTORIAN SYMPOSIUM

Geotechnical characterisation – managing design and construction risk

AUSTRALIAN GEOMECHANICS SOCIETY
VICTORIA CHAPTER